UNIVERSITY OF ILLINOIS
BULLETIN

Vol. 45 December 3, 1947 No. 23

ENGINEERING EXPERIMENT STATION
BULLETIN SERIES No. 365

EXPERIENCE IN ILLINOIS WITH JOINTS IN CONCRETE PAVEMENTS

A REPORT OF INVESTIGATIONS

CONDUCTED BY

A SPECIAL COMMITTEE COMPOSED OF
MEMBERS OF THE STAFFS OF THE DEPARTMENTS OF
CIVIL ENGINEERING AND THEORETICAL AND APPLIED
MECHANICS OF THE COLLEGE OF ENGINEERING

AND

THE DIVISION OF HIGHWAYS
STATE OF ILLINOIS

BY

JOHN S. CRANDELL
VERNON L. GLOVER
WHITNEY C. HUNTINGTON
J. DOUGLAS LINDSAY
FRANK E. RICHART

AND

CARROLL C. WILEY

PRICE: ONE DOLLAR
PUBLISHED BY THE UNIVERSITY OF ILLINOIS
URBANA

Published every five days by the University of Illinois. Entered as second-class matter at the post office at Urbana, Illinois, under the Act of August 24, 1912. Office of Publication, 358 Administration Building, Urbana, Illinois. Acceptance for mailing at the special rate of postage provided for in Section 1103, Act of October 3, 1917, authorized July 31, 1918.

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BY

JOHN S. CRANDELL
Professor of Highway Engineering, University of Illinois

VERNON L. GLOVER
Assistant Chief Highway Engineer, Illinois Division of Highways

WHITNEY C. HUNTINGTON
Head of Department of Civil Engineering, University of Illinois

J. DOUGLAS LINDSAY
Civil Engineer, Illinois Division of Highways

FRANK E. RICHART
Research Professor of Engineering Materials, University of Illinois

AND

CARROLL C. WILEY
Professor of Civil Engineering, University of Illinois

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EXPERIENCE IN ILLINOIS WITH JOINTS IN CONCRETE PAVEMENTS

I. Introduction

1. *Statement of General Problem.*—The practice of installing joints in concrete pavements, to provide for the movements which take place when temperature and moisture conditions change and to relieve the stresses which occur, is almost as old as the modern hard-surfaced road itself. A patent granted in 1871, covering the use of gum, tar, rubber, or other water-repellent materials as a filler for joints in pavements, indicates that the early designers were conscious of the need for joints. However, the use of joints has been by no means universal, nor have the practices been uniform. Many different materials and designs have been used throughout the country, and engineers have widely different ideas as to the spacing, size, and type of joint which best meets requirements.

An example of the diversity of the opinions which existed with respect to joints is shown in Table 1, which gives the jointing practices in effect throughout the United States on January 1, 1939. A more complete summary of State Highway Department specifications and practices in the design of portland cement concrete pavements, in effect at the beginning of the national war emergency in 1942, is given in Table 2 (see pocket attached to inside back cover). It will be seen from these tables that the practices of the various states with respect to joints were rather widely varied.

While joints were not adopted for general use by the Illinois Division of Highways until 1928, many miles of pavement have since been built in which joints of different types and materials were incorporated, and much experience has been gained. A large amount of investigational work has been done by the Division of Highways and the University of Illinois, both separately and cooperatively. The results of this work are recorded in individual departmental reports. However, in view of the importance of the problem of jointing pavements, it is believed that a report consolidating the studies of both agencies and recording the experience that has been gained by the installation of joints in pavements in Illinois will be of much value to highway designers.

2. *Historical Background of Practice in Illinois.* — During the early period of development of a state-wide system of hard-surfaced highways in Illinois, no attempts were made to provide for the expansion

TABLE 1

EXPANSION AND CONTRACTION JOINT SPECIFICATIONS IN EFFECT IN THE UNITED STATES, JANUARY 1, 1939

State	Date of Standard	Types of Expansion Joints	Spacing		Material	Load Transmission Device	Slab Reinforcement	Remarks
			Expansion joints, ft.	Contraction joints, ft.				
Alabama	1935	Poured Premolded	40.00	20.00	Bituminous	Dowels U-Bars	Bar mat	Little concrete paving done.
Arizona	1935	Poured Premolded	60.00	20.00	Bituminous	Dowels	None	Premolded joint in all new pavements.
Arkansas	1936	Poured Premolded Air-chamber	50.00	None used	Asphalt and other suitable materials	Dowels	Wire mesh	
California	1935	Poured Premolded	60.00	20.00	Bituminous Bituminous fiber	Dowels	See remarks	No reinforcement except where slab acts as beam.
Colorado	1937	Premolded	90.00	30.00		Dowels	See remarks	No reinforcement except where slab acts as beam.
Connecticut	1937	Premolded	75.75	25.25	Cork	Translode J-Bars	Bar mat or wire mesh as specified	Rubber or other fillers sometimes used.
Delaware	1936	Premolded	90.00	30.00	Various elastic materials	Dowels	See remarks	Use reinforcement for special conditions.
Florida	1937	Premolded	100.00	20.00	Cork Rubber Cork-rubber	Translode Dowels	Bar mat	
Georgia	1937	Premolded	90.00	30.00	Bituminous Cork Cork-rubber	Dowels	None	
Idaho	1936	Poured Premolded			Bituminous	Dowels	Wire mesh and edge bars	
Illinois	1938	Premolded	50.00	None used	Fiber Cork Rubber Cork-rubber	Dowels J-Bars Translode	Wire mesh	
Indiana	1936	Premolded	40.00	None used	Fiber Cork Rubber Bituminous	Crosslode Translode Dowels	Wire mesh	Use of contraction joints discontinued February, 1936.

NOTE: Tabulation based on information furnished by State Highway Departments and study of specifications of the various States. No responsibility is assumed for errors or omissions.

TABLE 1 (CONTINUED)
EXPANSION AND CONTRACTION JOINT SPECIFICATIONS IN EFFECT IN THE UNITED STATES, JANUARY 1, 1939

State	Date of Standard	Types of Expansion Joints	Spacing: Expansion joints, ft.	Spacing: Contraction joints, ft.	Material	Load Transmission Device	Slab Reinforcement	Remarks
Iowa	1937	Premolded	120.00	30.00	Bituminous	Dowels		
Kansas	1937	Premolded			Bituminous	Translode Dowels	None used	
Kentucky	1935	Premolded	90.00	30.00	Bituminous Rubber Cork	Dowels or as specified	Wire mesh	
Louisiana	1937	Premolded	90.00	30.00	Cypress Cork Fiber	Dowels	Bar mat Wire mesh	
Maine	1937	Premolded	40.00	None used	Bituminous	Dowels	Bar mat top and bottom of slab	
Maryland	1936	Premolded	90.00	30.00	Fiber	Dow-weld Ridgely	None	
Massachusetts		Premolded	57.00	None used	Cork Rubber	Dowels	Bar mat Wire mesh	
Michigan	1937	Premolded	60.00	30.00	Bituminous Cork Fiber	Dowels or others	Bar mat Wire mesh	
Minnesota								
Mississippi		Premolded	40.00	None used	Cork Fiber Bituminous	Dowels U-Bars Crosslode	Bar mat Wire mesh	
Missouri	1937	Premolded Air-chamber	100.00	50.00	Various	Translode J-Bars U-Bars	Bar mat Wire mesh	
Montana	1935	Premolded			Bituminous	Dowels	Wire mesh	Very little concrete paving done.
Nebraska								

TABLE 1 (CONTINUED)

EXPANSION AND CONTRACTION JOINT SPECIFICATIONS IN EFFECT IN THE UNITED STATES, JANUARY 1, 1939

State	Date of Standard	Types of Expansion Joints	Spacing		Material	Load Transmission Device	Slab Reinforcement	Remarks
			Expansion joints, ft.	Contraction joints, ft.				
Nevada								No concrete paving done in past six years and none contemplated.
New Hampshire	1935	Poured Premolded	50.00	None used	Bituminous	Dowels	Bar mat	
New Jersey	1935	Premolded	53.00	None used	Bituminous Cork Others	Channel dowels	Bar mat	
New Mexico								No concrete paving done for several years and none contemplated.
New York	1935	Poured Premolded	94.50 max.	None used	Bituminous Cork Wood and others	Acme	Bar mat	
North Carolina	1937	Poured Premolded	90.00	30.00	Bituminous Cork Rubber	Dowels	In bridge approaches only	Cork-rubber filler sometimes used.
North Dakota								No concrete pavements being constructed or contemplated.
Ohio								
Oklahoma		Premolded	30.00	None used	Bituminous Fiber	Dowels	Edge bars only	Have tried some of air-chamber types but find cost prohibitive.
Oregon	1937	Premolded	90.00	15.00	Bituminous	Dowels	In bridge approaches only	
Pennsylvania	1937	Premolded			Various	Dowels Dow-weld J-Bars U-Bars Beth. Translode	Bar mat Wire mesh	
Rhode Island	1937	Premolded	73.50	36.75	Bituminous Cork Rubber	Dowels	Wire mesh	Contraction joints only when specified.

TABLE 1 (CONCLUDED)

EXPANSION AND CONTRACTION JOINT SPECIFICATIONS IN EFFECT IN THE UNITED STATES, JANUARY 1, 1939

State	Date of Stand-ard	Types of Expansion Joints	Spacing		Material	Load Trans-mission Device	Slab Reinforcement	Remarks
			Expan-sion joints, ft.	Con-traction joints, ft.				
South Carolina	1937	Poured Premolded	90.00	30.00	Bituminous	Translode	Bars and wire mesh over culverts	Use of other types of joints permitted on approval.
South Dakota		Premolded Air-chamber	90.00	30.00	Cork	Dowels	Wire mesh Bars at bridge approaches only	
Tennessee	1937	Poured Premolded Air-chamber	90.00	30.00	Bituminous Fiber Rubber	Dowels J-Bars U-Bars	Bar mat or wire mesh as specified	
Texas		Poured	78.50	26.17	See remarks	Dowels	Shear bars only	Sawdust or suitable material mixed with oil and sealed with tar.
Utah		Premolded	90.00	30.00	Bituminous	Dowels	None	
Vermont	1937	Premolded	55.33	11.00	Bituminous Cork	Dowels	Bar mat	Layout of bar mat is an elaborate system.
Virginia	1935	Premolded	90.00	30.00	Cork Rubber	Dowels	Wire mesh	
Washington		Premolded	60.00	15.00	Bituminous	Dowels	See remarks	No reinforcement in slab when adequate subgrade conditions exist.
West Virginia	1935	Poured Premolded	93.00	31.00	Bituminous Wood	Dowels	Wire mesh and marginal bars when specified	
Wisconsin	1937	Premolded	30.00	None used	Cork Rubber and Others	Dowels	Bar mat Wire mesh	A tamped cork and asphalt filler sometimes used.
Wyoming	1936	Premolded	50.00	None used	Cork	Dowels	Yes	Very little concrete paving done.
Washington, D.C.		Air-chamber	60.00 40.00	30.00 None used		Dowels J-Bars	Wire mesh	Spacing of expansion joints dependent on temperature.

FIG. 1. TYPICAL BLOWUP IN CONCRETE PAVEMENT

and contraction of concrete pavements. There were two reasons for this practice. First, it was believed that the cracks, which were known to occur in long, thin, concrete slabs subject to wide and sometimes sudden changes in climatological conditions, could be adequately sealed with asphalt cement to prevent the concrete from spalling at the edges of the cracks and to prevent also the entrance of dirt into the cracks. Second, it was generally believed that normal changes in length of the pavement would be absorbed by its numerous horizontal curves.

Experience later proved that cracks could not be sealed with any of the available fillers against the entrance of foreign materials. Each time the pavement contracted, dirt entering the cracks prevented them from fully closing again on subsequent expansion. The action was progressive, the cracks becoming a little wider with each cycle of expansion and contraction. This eventually resulted in an actual lengthening of the pavement which was not absorbed by the changes in alignment, as was expected, and, as a consequence of the concentration of compressive stresses during periods of expansion, failures of the concrete commonly known as blowups occurred. Figure 1 is a photograph of a typical blowup.

The frequent occurrence of blowups finally became such a serious problem, from the standpoint of highway safety and pavement maintenance costs, that the various highway districts adopted the general practice of cutting transverse joints at those locations where blowups appeared imminent. At locations where blowups did occur, space for future expansion of the concrete was provided at the time the failure was repaired by leaving an opening in the slab and filling it with asphalt.

In 1923 one of the highway districts, as an experiment, began constructing 4-in. open joints in a few sections of concrete pavement, spacing the joints nearly one-half mile apart. This spacing, however, was gradually decreased, until in 1926 it averaged a little less than 1,000 ft. These open joints apparently were useful in delaying the occurrence of blowups, and in 1928 their use was adopted as standard practice throughout the state. They were spaced at not less than 800 ft. nor more than 1,000 ft. While a limited mileage of pavement was constructed in the Chicago area during 1929 and 1930 without open joints, by 1931 they were being built into all pavements constructed directly by the state and by the counties under state supervision. In constructing the joints, the depth of the slab adjacent to the opening was increased to offset the dynamic effect of wheel loads passing over the joint.

By 1932, however, it was discovered from examinations of the earlier joints of this type that they were closing rapidly, making it necessary at the end of four or five years to widen many of the joints or cut new ones to prevent blowups. Certain features of this type of joint were undesirable from the standpoint of design. It was quite common for the slab to break transversely at or near the transition from the strengthened portion to the standard central depth; the extrusion of the asphalt used as a filler formed a high ridge over the joint, which had to be removed several times a year to prevent excessive impact under heavy wheel loads; when the pavement contracted, there was not enough aspalt to fill the joint and more had to be added; dirt entered the expansion space from the subgrade, working up into the joint and displacing the filler; and, in some instances, the transverse crack which occurred near the joint acted a great deal like a blowup. In these latter cases, the slab rose several inches, making it necessary to cut the concrete in order to restore the slab to its proper position.

In view of these conditions, the Chief Highway Engineer, on February 21, 1931, appointed a committee composed of four bureau chiefs and one district engineer to make a comprehensive investigation and submit recommendations as to the future policy which should be followed.

Under the direction of this committee, a study of the concrete pavements constructed from 1922 to 1931, inclusive, was made. The data collected were analyzed and the committee submitted its findings and recommendations to the Chief Highway Engineer on February 1, 1932. The investigation revealed that during the period mentioned, 4,633 miles of pavement had been constructed without expansion joints, that 3,940 joints had been cut in these pavements, and that up to the end of 1931, 3,400 blowups had occurred on the 4,633 miles. The study showed that the rate at which blowups occurred increased with the age of the pavement; for example, pavements built in 1922 averaged two blowups per mile in 1931.

During the same period (1922-1931, inclusive), 3,417 miles of pavement were constructed with 4-in. open joints, of which only 269 miles, all in one district, had been constructed prior to 1928. In the pavements built prior to 1928, the ends of the slabs adjacent to the joints were not strengthened by edge thickening, a procedure which was followed in the pavements built during and after that year. The data assembled by the committee showed that the 4-in. joints closed at the rate of approximately one inch per year, and that blowups could be expected about the fifth year after construction, although scattered blowups might occur earlier. This conclusion was based not only on measurements of the width of the joints in pavements of various ages, but also on data as to the occurrence of blowups and the necessity of widening existing joints and cutting new joints to keep blowups to a minimum. That blowups may occur early in the life of a pavement, in spite of 4-in. joints, was demonstrated by the fact that five blowups occurred in one district during the year the pavement was built.

The committee agreed that expansion space of some kind should be provided, but did not feel that the data available justified definite recommendations as to particular types. While the 4-in. joints were not considered satisfactory, it was recommended, in the absence of a more suitable type, that their use be continued during 1932. In the meantime, it was thought advisable to construct a number of experimental sections in which joints with smaller openings spaced at shorter intervals would be provided. It was suggested that both 1-in. joints spaced 200 ft. apart and 2-in. joints spaced 400 ft. apart be used, and also that 4-in. joints, preferably of the mechanical type, be provided every 1,000 ft. It was felt that the strengthening of the slab ends adjacent to joints should be accomplished by some means other than the edge thickening then in use.

Experimental installations of several types of joints were made during the late summer and fall of 1932, substantially as recommended by the committee. The various types installed were obtained by inviting manufacturers of joints to submit their particular types or designs. The bituminous premolded type was also included. Approximately 30 miles of pavement containing these experimental joints were constructed, and periodic inspections were made to observe their behavior.

On the basis of the knowledge gained from these trial installations, the Division of Highways, in 1933, chose the copper-sealed, all-metal, air-chamber joint as the type most nearly meeting the known requirements for extending the service life of pavements, eliminating unsafe conditions caused by frequent blowups, and reducing maintenance costs of the pavements. It was expected that such joints would prevent infiltration of foreign material into the expansion space and, if closely spaced, would be effective in reducing the number of transverse cracks, thus preserving over a long period most of the expansion space originally provided.

After a study of transverse crack intervals in existing concrete pavement, it was decided to use, at intervals of 90 ft., copper-sealed, all-metal, air-chamber expansion joints with not less than ¾ in. nor more than 1 in. of free expansion space, to insert two copper-sealed, metal contraction joints between each pair of expansion joints, thus providing a spacing of 30 ft. between joints, and to omit the marginal bars previously used. The use of dowel bars ¾ in. in diameter and 24 in. in length, spaced not less than 12 in. nor more than 15 in., center to center, or some other approved load transmission system, was specified, provisions being made to break the bond between the bars and the concrete so as not to hinder the free movements of the slab.

The standard practice governing the use of joints conformed with the requirements of the U. S. Bureau of Public Roads (now Public Roads Administration) for Federal-aid construction. These requirements were set forth in a memorandum to District Engineers (U. S.) March 3, 1934, signed by the Chief of the Bureau, specifying that:

"(1) Expansion joints be used at intervals not greater than 100 feet, providing expansion openings of not less than ¾ inch nor greater than 1 inch.

"(2) Provisions be made for load transfer across expansion joints, either by the use of dowel bars or other devices which will accomplish the same purpose.

"(3) Provision be made for crack control by the use of steel reinforcement, or suitably designed transverse contraction joints or planes of weakness so spaced that the distance between such contraction joints does not exceed 30 feet."

In addition to these definite requirements, the memorandum contained a number of explanatory statements and recommendations relating to provisions for transmission of load across the joints and to the necessity of sealing the joints against infiltration of inert material and the entrance of surface water into the joints. With proper spacing of joints, the usual marginal bars were not required.

From 1933 to 1937, inclusive, the period during which metal joints were used, the Division of Highways approved for general use three designs of metal-sealed, all-metal, air-chamber expansion joints and an equal number of metal-sealed contraction joints. Those approved are identified herein as Types J-1, J-2, J-4, and J-5, the latter two being substantially of the same design.

The J-1 joints were included in the experimental installation in 1932, further improved during 1933, and used until metal joints were discontinued.

The J-2 joints were developed during 1933 and trial installations were made in September of the same year. These joints were approved for use in July, 1934. After they had been incorporated in a considerable mileage of pavement, however, it became apparent that satisfactory installations were not being obtained because of certain defects inherent in the design, and approval was withdrawn in February, 1935.

The J-4 joints were developed during 1934; trial installations were made in September of that year and the joints were approved for use during January, 1935. They were used until February, 1938, when metal joints were discontinued.

The J-5 joints, quite similar to the J-4 joints, were approved in April, 1936, and used until metal joints were discontinued.

The J-1, J-4, and J-5 joints utilized ¾-in. dowel bars, 24 in. in length, for the transmission of load across the joints; the J-2 joints incorporated a load transmission device identified herein as Type L-2. Later on, the L-1 load transmission device, which consisted of a short dowel inserted in metal sleeves bonded in the concrete on opposite sides of the joint, was developed. This device, after having been installed experimentally in September, 1935, with good results, was approved as an alternate for the conventional dowel bars. Descriptions of the joints and load transmission devices approved for use are given in Chapter II (see pages 29-58).

The use of metal joints was discontinued at the end of the 1937 construction season, because it was found, as will be discussed later, that serious defects were developing in these joints and the anticipated results were not being obtained. Experience also proved that the practice of installing joints at 30-ft. intervals does not control transverse cracking. The metal joints, however, appeared to have been effective in postponing for an indefinite period the occurrence of blowups. With these facts in mind, the Division of Highways in February, 1938, adopted the practice of installing $\frac{3}{4}$-in. premolded joints of fiber, cork, rubber, or cork-rubber at 50-ft. spacings. Knowing that transverse cracks would occur, reinforcement was provided to keep such cracks closed so that inert material could not enter. The use of the types of load transmission devices employed in connection with the all-metal joints — namely, the conventional dowel, and the L-1 and L-2 devices — was continued.

With the exception of the earliest pavements in which 4-in. joints were built, all of the pavements with transverse joints were of the thickened edge design with metal longitudinal center joint and $\frac{1}{2}$-in. round deformed steel tie bars. Practically all of the pavements with 4-in. joints were of the 9-6-9 design; that is, 9 in. thick at the edges, tapering in 2 ft. to the uniform central thickness of 6 in. Most of the pavements with air-chamber joints and premolded joints were of the 9-6$\frac{1}{2}$-9 or 9-9-7-9-9 design, although some built in the metropolitan areas were of the 10-10-8-10-10 design. Marginal bars, $\frac{3}{4}$ in. or $\frac{7}{8}$ in. in diameter, were placed 6 in. from the edges and at the mid-depth of the slab in all pavements with 4-in. joints for the purpose of furnishing mutual support between adjacent slabs at transverse cracks which formed. When all-metal joints at close intervals were adopted, it was decided that these marginal bars could be omitted, on the theory that transverse cracks would seldom occur in the short panels formed between the joints. With the adoption of premolded joints, the pavements were reinforced with wire fabric weighing approximately 55 lb. per 100 sq. ft.

The proportions used for the concrete in pavements built prior to 1930 were 1:2:3$\frac{1}{2}$ by volume; after that time the concrete mixtures were designed by the mortar voids method to give a strength of at least 3,500 lb. per sq. in. in compression and 650 lb. per sq. in. in flexure at 14 days. Aggregates and cement were obtained from approved commercial plants in Illinois and surrounding states.

Aggregates for use in highway construction in Illinois are obtained principally from approximately 150 commercial sources of sand, gravel, and crushed stone, located largely in the north central and northern parts of the state and along the boundary rivers in the southeastern and southwestern parts of the state.

Practically all of the stones used in Illinois were probably formed by the organic process and are classed as either limestone or dolomite. The limestones are composed mostly of calcium carbonate. The dolomites are composed of calcium carbonate and magnesium carbonate.

The sands and gravels, which are of glacial origin, in some cases are stream borne, especially in the southern portions of the state. They consist mainly of limestone and dolomite particles, although some of the river sands contain relatively large percentages of silicious material.

In general, the aggregates used in Illinois may be said to be of fairly good quality; little trouble has been encountered which could be attributed to unsound aggregates. While aggregates bear an important relation to the performance of joints, no attempt was made in these investigations to correlate the action of joints with the properties of the aggregates used. It is believed that the scope of the investigations was so broad, covering construction extending over six years in all parts of the state, that the influence of different aggregates can be disregarded in overall comparisons between joints.

All materials were subject to inspection and test under the provisions of the "Standard Specifications for Road and Bridge Construction" and special provisions in force at the time of construction.

The soils in Illinois vary from the more friable sandy and silty loams to the more plastic sandy and silty clay loams, clay loams, and clays. The silty loams predominate. Comparatively small areas of sand and loess are distributed throughout the state. A few peat bogs are encountered in the northeastern portion of the state.

No attempt was made in these investigations to correlate the action of joints with the types of soils on which they were located. As in the case of aggregates, it is believed that the scope of the investigations was so broad that variations in soils would not be reflected in the overall comparisons. It is hoped that the effects of soils and aggregates will be considered in later investigations.

Generally speaking, the pavements were built upon existing soils with little being done to treat or replace the soils to improve the subgrade; the technique of soils testing was not developed to a point which would enable engineers to predict with sufficient assurance the behavior of all types of soils under anticipated conditions. The Division of Highways felt that under such circumstances it was more economical to correct those relatively few conditions which would develop as the result of faulty subgrade than to try to anticipate them and treat and replace subgrade material before construction. Exceptions to this policy were made in a few cases where the existing soil conditions were so obviously bad that it was definitely apparent that the subgrade would not provide adequate support for the pavement slab. In these cases the subgrade material was treated or replaced. Drainage of the pavement and subgrade was in general accomplished by sloping shoulders and side ditches. On hills and grades curb and gutter were sometimes built along the edges of the pavement to carry the water to an outlet at a low point from which it was carried to the side ditch. In a few cases special drainage systems were installed at the time of construction to drain the subgrade at locations known to be excessively wet or to intercept a flow of ground water before it reached the subgrade.

3. *Scope of Present Study.*—The decision of the Illinois Division of Highways to discontinue the use of all-metal joints and to adopt the practice now in effect was based on a number of field and laboratory investigations made during the period these joints were in use. These investigations, particularly the later ones, will be discussed in this bulletin, and their results summarized and analyzed.

Five major field investigations were made during the period from 1934 to 1939, inclusive. The first of these, conducted in 1934 and 1935, dealt with the J-2 joints, various difficulties having been experienced with this type without successful methods of correction being developed.

A fairly extensive investigation of J-1 and J-4 joints was made in 1935 and 1936, primarily to secure data which would be of value to the construction organization in improving the methods of installation of the joints.

During the summer of 1937, an extensive investigation was started by the Division of Highways to study the performances of all types of all-metal joints which had been installed in Illinois pavements up to that time.

Coincident with this investigation, but entirely independent of it, an extensive investigation of expansion and contraction joint practices in Illinois was made by a committee, composed of members of the staffs of the Departments of Civil Engineering and Theoretical and Applied Mechanics of the College of Engineering, appointed by the President of the University of Illinois at the request of the Governor of Illinois. The University committee also was instrumental in arranging for an experimental section, hereinafter called the Armington Experimental Road, in which various types of joints and load transmission devices submitted to the committee by manufacturers were installed. This work was carried on with the cooperation of the Division of Highways under the direction of the University committee. A complete report of this project is included in this bulletin (see pages 197–243).

In November, 1939, the Director of the Department of Public Works and Buildings ordered a thorough investigation of the condition of all types of joints which had been incorporated in pavements in Illinois, to determine beyond any question of doubt whether all-metal joints were or were not suitable for use. This investigation was conducted by the personnel of the Division of Highways, with the University committee previously mentioned acting in a consulting capacity.

Laboratory investigations were carried on by both the Division of Highways and the University of Illinois. The Division of Highways laboratory investigation started when all-metal joints were introduced, and was carried on more or less continuously during the period when all-metal joints were used. Since the principal purpose of the laboratory tests was to determine whether a joint or load transmission device submitted for approval possessed defects which might make it undesirable, this investigation did not follow any definite program, but was conducted as samples were submitted by manufacturers or their agents.

The laboratory investigation by the University was conducted simultaneously with the field investigation and under the direction of the same committee. It consisted of a highly organized program of tests of most of the joints and load transmission devices which had been tested previously by the Division of Highways and of a number of others which had been submitted at the invitation of the University committee.

A supplementary field investigation was made early in 1943, after the preparation of the manuscript for this bulletin had been started, to obtain further information on fiber joints, which had been installed for only one year when examined in 1939, and to study the relative effect of joints and transverse cracks in contributing to surface roughness.

4. *Object of the Bulletin.*—It is the object of this bulletin to present the experiences in Illinois in connection with the use of joints in concrete pavements, particularly during the period from 1928 to 1940. When reference is made in a few instances to experiences earlier or later than this period, specific dates are given. The bulletin includes descriptions of joints and load transmission devices which have been used in Illinois and elsewhere in the United States; a study of current

practices in the United States; a discussion of the laboratory tests which have been made; reports of the field investigations; a report of the Armington Experimental Road; and a report of the work done by the University in cutting joints with abrasive wheels. The data obtained from the investigations, and the knowledge gained through years of experience in building thousands of miles of pavements with and without expansion joints, are summarized, and finally conclusions are drawn, based on the experiences in Illinois, which it is hoped may be of value to highway engineers.

This bulletin is based entirely on experiences in Illinois. While members of the University committee discussed with highway engineers in other states and the Public Roads Administration the general problem of jointing pavements, their conclusions were drawn only from their findings in Illinois. Other states may have had similar experiences, but the statements made herein apply only to Illinois.

5. *Acknowledgments.* — This bulletin is published by the Engineering Experiment Station of the University of Illinois, of which DEAN M. L. ENGER is Director, under a cooperative arrangement with the Illinois Division of Highways, of which W. W. POLK is Chief Highway Engineer.

The bulletin was prepared by J. D. LINDSAY, Civil Engineer in the Division of Highways, under the direction of PROFESSOR W. C. HUNTINGTON, Head of the Department of Civil Engineering of the University of Illinois, V. L. GLOVER, Assistant Chief Highway Engineer, Division of Highways, and H. W. RUSSELL, Engineer of Materials, Division of Highways.

The field investigation by the University of Illinois was conducted by a committee consisting of PROFESSOR HUNTINGTON, Chairman, PROFESSORS J. S. CRANDELL, F. E. RICHART, and C. C. WILEY, appointed by A. C. WILLARD, then President of the University. This committee arranged for the experimental installation of joints and load transmission devices in the Armington Experimental Road. The installations were made under the committee's direction in cooperation with the Division of Highways. R. G. OESTERLE, Resident Engineer for the Division of Highways on the Armington Experimental Road, rendered valuable service to the University committee both during and after completion of construction.

The laboratory work by the University committee was supervised by T. J. DOLAN, then Assistant Professor in Theoretical and Applied Mechanics, under the direction of PROFESSOR RICHART.

Experimental work on cutting joints with abrasive wheels was conducted by E. S. FRASER and WARREN GRASSO under the direction of PROFESSORS HUNTINGTON and CRANDELL. The cutting machine was designed by ALBERT KRIVO. Much of the material for Chapter VI was taken from a thesis prepared by MR. FRASER.

The field investigations and laboratory tests by the Division of Highways were under the direction of V. L. GLOVER, formerly Engineer of Materials, now Assistant Chief Highway Engineer. J. D. LINDSAY analyzed the data from the field investigations, wrote the departmental reports, and supervised the laboratory tests. The work of preparing reports of laboratory tests was shared by O. LARSEN, Civil Engineer in the Bureau of Materials, and MR. LINDSAY.

The following employees and former employees of the Illinois Division of Highways deserve credit for their services in conducting tests and collecting field data, and in the reduction of data: W. F. BARNEY, G. M. BOHLIG, W. J. BOLLING, F. P. BROCK, E. R. CLEMMONS, J. S. COOPER, C. E. CULLEN, E. H. CULPEPPER, J. I. DALRYMPLE, R. F. ENGEL, G. W. HARNEY, B. L. JAKAITIS, R. E. KELLEY, D. KUYKENDALL, L. E. LEKA, R. A. LONIER, E. O. MANON, J. M. MARTIN, M. D. REED, E. R. REIFLER, M. E. RIDGE, I. C. SAWYER, L. WALKER, C. F. WINTZ, H. K. DOLBOW, and M. F. ZOGG.

Acknowledgment is made of the courtesy of the late R. R. ZIPPRODT, Research Engineer, American Iron and Steel Institute, in permitting the material from their tabulation of State Highway Department specifications and practice in the design of portland cement concrete pavements to be reproduced herein.

II. Description of Types of Joints and Load Transmission Devices

6. *General Statement.* — The types of joints and load transmission devices described herein include those approved for general use in Illinois, those which have been tested by the University of Illinois and the Division of Highways, and others whose use throughout the United States has been sufficient to make them of general interest to highway engineers. These descriptions include photographs, drawings, and such written comments as are deemed necessary to give the reader a clear understanding of these joints and devices.

All joints and load transmission devices tested by the Division of Highways and the University of Illinois and installed in the Armington Experimental Road were submitted by the manufacturer of the product and were tested and used with his approval.

7. *Expansion and Contraction Joints.* — Expansion joints fall into three general classifications; namely, metal-sealed joints, open joints with poured fillers, and joints with preformed fillers. While an expansion joint also may be considered as a contraction joint because it performs the same function, strictly speaking a contraction joint is one which provides only for contraction. Contraction joints fall into two general classes: the metal dividing plate type and the dummy type. The individual joints in these several classes are discussed below:

(a) All-Metal Expansion Joints

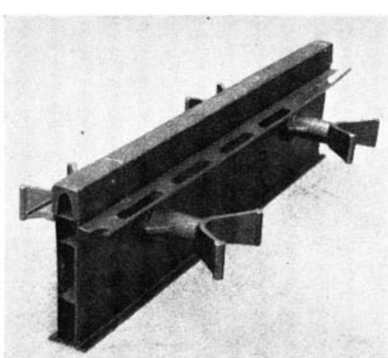
Metal-sealed expansion joints may be further divided into two subclassifications — the so-called air-chamber joint, and those joints which have a metal seal over a preformed filler.

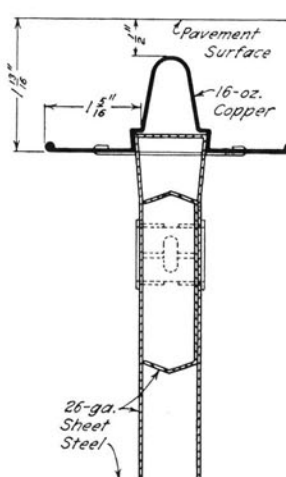
In general, the air-chamber joint consists of a box or form made from light-gage sheet steel to the form and general dimensions of the cross section of the pavement. A separator, such as a channel shaped member made from light-gage steel, is usually provided to prevent the side walls from collapsing under the pressure of the plastic concrete. The top and ends of the box are covered by a seal of corrosion-resistant sheet metal having flanges which extend into the concrete on either side of the joint. The seals are made of various shapes designed to permit changes in the joint width without producing excessive stress in the metal. All the air-chamber joints included in this study had seals made from sheet copper weighing approximately 16 oz. per sq. ft. The seven types of air-chamber joints discussed in this bulletin are described below.

(1) J-1 Expansion Joint. Figure 2 is a photograph of a J-1 expansion joint, showing one of the later designs. The copper seal on this joint was made in the shape of an inverted "U". The seal was carried down over the ends of the joint, the junctions between top and end seals being lapped and soldered. A premolded bituminous cap cemented over the copper seal served to protect the seal during installation and to provide a guide for edging. The premolded cap was left in place after installation to protect the seal from the action of traffic. Figure 3 is a cross section of a joint for a 7-in. pavement, showing some of the details of design. This joint, first used in Illinois in experimental installations made during 1932, was approved for general use in 1933. It was included in the program of tests at the University of Illinois and was installed in the Armington Experimental Road.

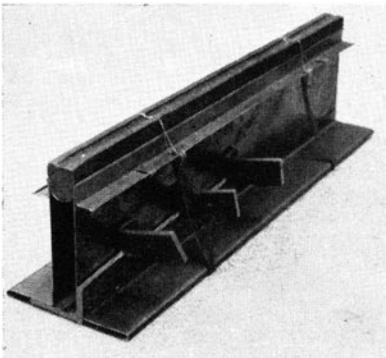
(2) J-2 Expansion Joint. Figure 4 shows a short section of the J-2 expansion joint, which had a U-shaped seal which rested on the side plates, and was held in place by wires or bands passing under the bottom of the joint and fastened to the flanges of the seal. A bituminous premolded cap over the copper seal is intended to be left in place after installation. Since experience in Illinois, however, showed that this cap was not satisfactory, it was not generally used. Instead, the joint was equipped with a steel installation bar which was removed as soon as the concrete had set up sufficiently to be edged. The space above the seal was later filled with a poured asphalt filler.

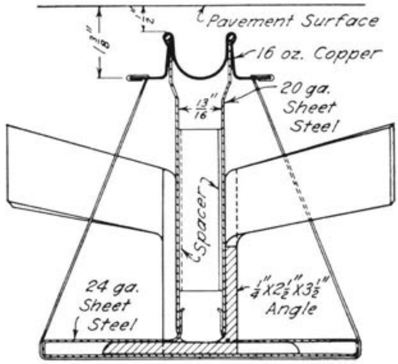
The joints used in Illinois did not have copper end seals. The ends were sealed by means of a ¼-in. steel plate attached to one side of the joint and enclosed in a galvanized sheet steel sleeve attached to the other side of the joint. A particular feature of this joint was the load transmission device furnished as a part of the joint, consisting of sections of ¼-in. x 2½-in. x 3½-in. structural angle 1 ft. in length, set alternately on each side of the

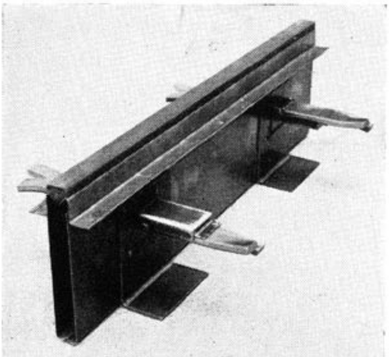


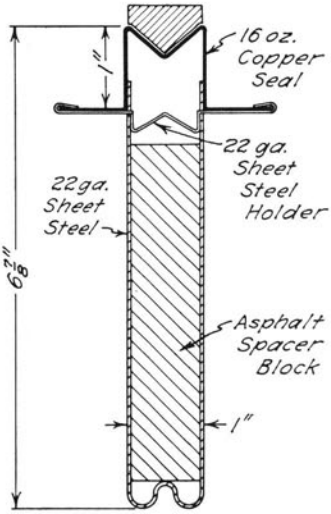


Figs. 2 and 3. J-1 Expansion Joint.
Fig. 2 (at Left) Shows the Joint with L-1 Load Transmission Device









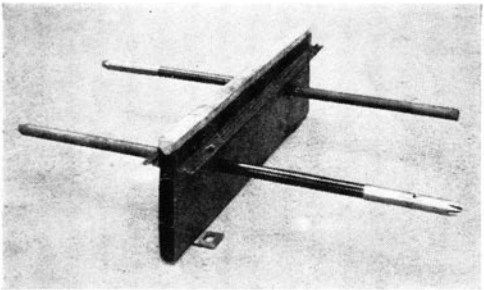
Figs. 4 and 5 (Above). J-2 Air-Chamber Expansion Joint

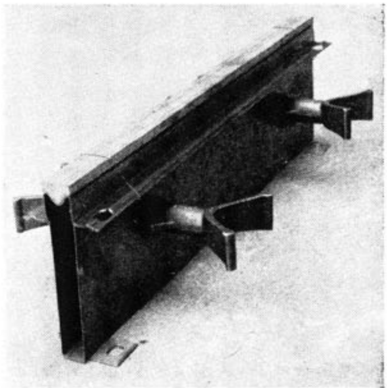
Figs. 6 and 7. J-3 Expansion Joint. Fig. 6 Shows the Joint with L-5 Load Transmission Device

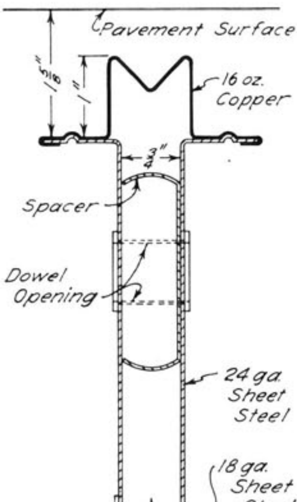
joint, with the vertical leg extending up along the side walls and the horizontal leg extending under the joint and the edge of the concrete slab adjacent to the joint. Three lugs, cut and formed from the vertical leg, extended outward from the side of the joint to provide anchorage in the concrete. Details of this joint, which was approved for general use in Illinois in July, 1934, are shown in Fig. 5. It was included in the program of tests at the University of Illinois and installed in the Armington Experimental Road.

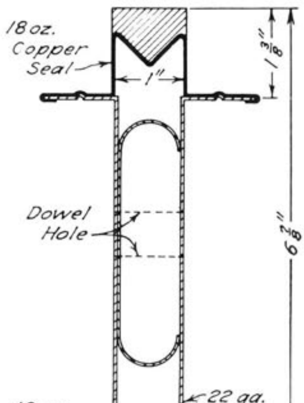
(3) J-3 Expansion Joint. Figure 6 shows the metal expansion joint developed for use in conjunction with the L-5 load transmission device, which will be described later. Figure 7 is a cross section of this joint showing details of design. This joint was not used generally in Illinois, but it was included in the program of tests at the University of Illinois and installed in the Armington Experimental Road under the direction of the University committee.

(4) J-4 Expansion Joint. The J-4 expansion joint, which was equipped with an M-shaped copper seal, is shown in Fig. 8. The seal was carried down over the ends, and the junctions between top and end seals were lapped and soldered. A wood installation strip was provided to protect the seal during installation and to serve as a guide for edging. This strip was later removed and the space above the seal poured with an asphalt filler. Figure 8 shows the joint equipped with conventional ¾-in. dowel bars 24 in. long, but the joint would accommodate other types of load transmission devices. Details of design are shown in Fig. 9. This joint was approved for general use in Illinois in January, 1935. It was included in the program of tests at the University of Illinois and installed in the Armington Experimental Road.



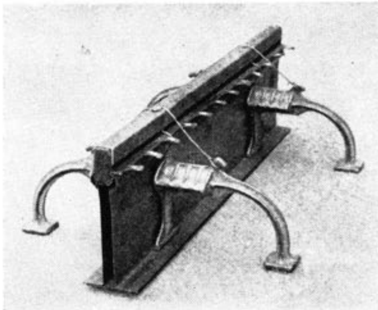


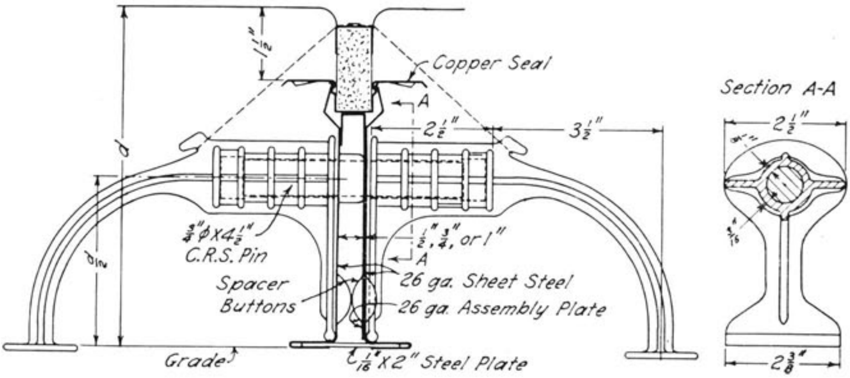




Figs. 8 (Above, at Left) and 9 (Below, at Left). J-4 Expansion Joint
Figs. 10 (Above, at Right) and 11 (Below, at Right). J-5 Expansion Joint.
Fig. 10 Shows the Joint with L-1 Load Transmission Device

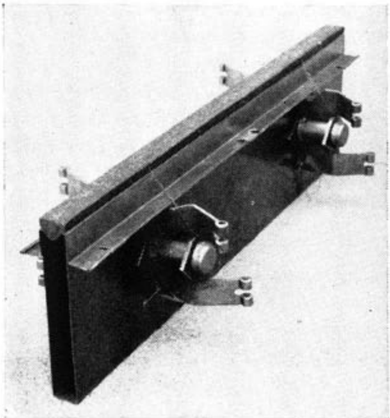
FIGS. 12 (AT RIGHT) AND 13 (BELOW). J-6 EXPANSION JOINT WITH L-4 LOAD TRANSMISSION DEVICE

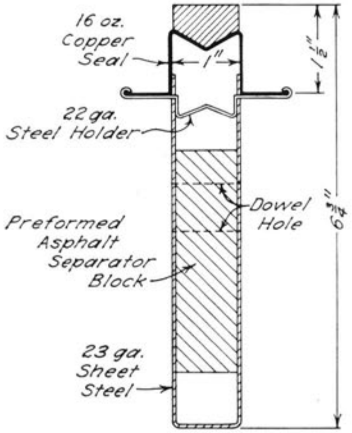




(5) J-5 EXPANSION JOINT. A short section of a J-5 expansion joint is shown in Fig. 10. This joint was almost identical in design and construction to the J-4 expansion joint. Figure 10 shows the joint equipped with the wing anchor load transmission device, but it was designed to accommodate other types. Figure 11 is a cross section of this joint showing details of design. Approved for general use in Illinois in April, 1936, this joint was included in the program of tests at the University of Illinois and installed in the Armington Experimental Road.

(6) J-6 EXPANSION JOINT. Figure 12 shows a short section of the J-6 expansion joint, which was provided with a U-shaped copper seal held in place between the side walls of the joint by means of the pressure exerted by a sponge rubber cap, fitted into the trough of the seal, and a rolled bead on each side of the seal which fits into a similar deformation along the top edge of each side plate. The copper seal was continuous over the top and ends of the joint, the seal being drawn on a 3-in. radius at the upper corners and carried down the ends to the bottom of the joint. The flanges of the copper seal extended only a short distance into the concrete, except for small lugs spaced at regular intervals along the seal. While Fig. 12 shows the joint equipped with the manufacturer's own load transmission device, this joint will accommodate other types. Figure 13 shows the details of design.





FIGS. 14 AND 15. J-7 EXPANSION JOINT. FIG. 14 (AT LEFT) SHOWS THE JOINT AND L-10 LOAD TRANSMISSION DEVICE

Since this joint provided a free expansion space of less than ½ in., it did not conform in this respect to the requirements of the Illinois Division of Highways or the Public Roads Administration. Installed experimentally in Illinois but not adopted for general use, it was included in the program of tests at the University of Illinois and installed in the Armington Experimental Road.

The manufacturer recommended the use of ½-in. expansion joints without intermediate contraction joints, and part of the installation in the Armington Road was in accordance with this recommendation. In addition, three 1-in. expansion joints and six contraction joints, made especially for the purpose, were installed on this project.

(7) J-7 EXPANSION JOINT. The J-7 expansion joint is shown in Fig. 14. On this joint the copper seal, which was M-shaped and very similar to others of this type, was mitered and soldered and carried down over the ends of the joint. A particular feature of this joint was that the separators, instead of being made from steel, consisted of blocks of premolded asphalt filler placed between the side walls of the joint. Figure 14 shows the joint equipped with a modified wing anchor load transmission device, L-12, which will be described later; however, the J-7 joint was designed to accommodate other types of load transmission devices. Details of design are shown in Fig. 15. This joint was not approved for use by the Division of Highways, but was included in the program of tests at the University of Illinois and installed in the Armington Experimental Road.

(b) Metal-Sealed Expansion Joints with Premolded Fillers

These joints consisted essentially of a premolded filler of one of the various types cut to the form and dimensions of the cross section of the pavement, with a copper seal made from 16-oz. sheet copper attached to the top and ends of the preformed filler. The three joints of this type included in this bulletin are described below:

(1) J-8 Expansion Joint. Figure 16 is a photograph of a short section of this joint, which consisted essentially of a premolded fiber-board body, copper top and end seals, a flanged channel black iron stiffener along the bottom of the fiber board, and a premolded bituminous cap, rectangular in shape, set on top of the seal and held in place by means of steel clips spaced at regular intervals. The end seals were of the same construction as the top seal and the junction between the two was mitered and soldered.

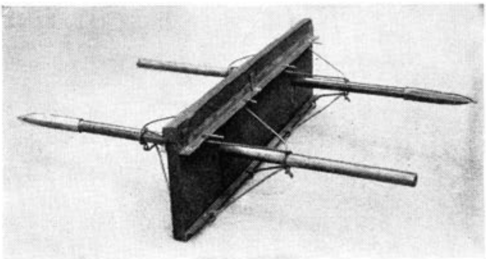


Fig. 16. J-8 Expansion Joint

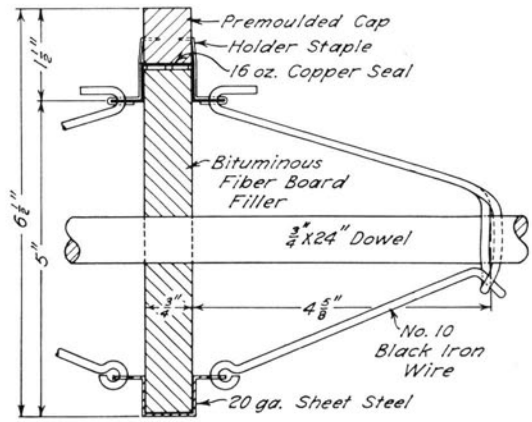
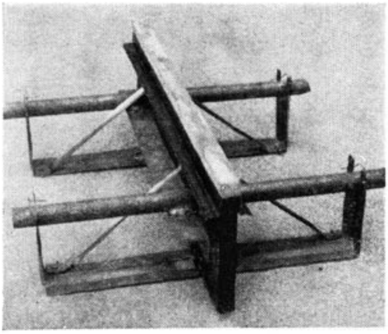
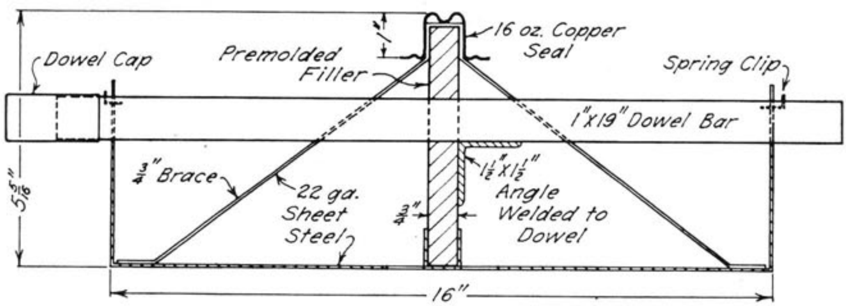


Fig. 17. J-8 Expansion Joint

This joint, which was designed to use the conventional ¾-in. dowel bar 24 in. long, was provided with dowel bar supports made from steel wire (see Fig. 17). Tested by the Division of Highways but not approved for general use, this joint was included in the program of tests at the University of Illinois and installed in the Armington Experimental Road.

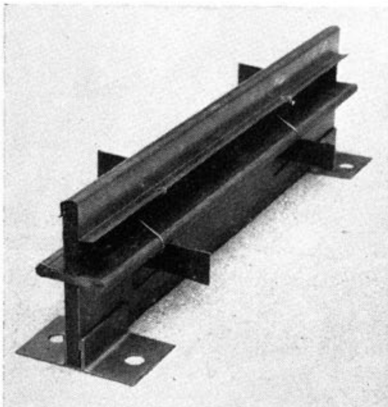


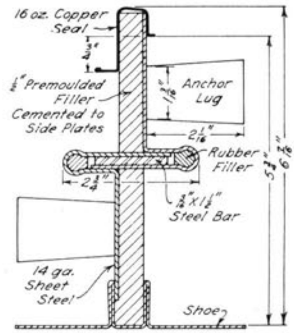
FIGS. 18 (AT LEFT) AND 19 (BELOW). J-9 EXPANSION JOINT WITH L-13 LOAD TRANSMISSION DEVICE



(2) J-9 EXPANSION JOINT. The J-9 expansion joint, shown in Fig. 18, consisted essentially of a premolded fiber joint filler cut to the form and approximate dimensions of the cross section of the pavement, copper top and end seals, and an especially designed arrangement for supporting 1-in. round dowel bars furnished with the joint. The copper seal, made in the form of an M with a very shallow trough, was provided with short flanges to anchor it into the concrete. These flanges had no deformations or other provisions for increasing the mechanical bond between the seal and the concrete. The end seals were of the same design and construction as the top seal, with the junction between them made by lapping the top seal over the end seal and covering the corner with a strip of electrician's friction tape. The steel dowel bars, 1 in. in diameter and 19 in. long, were welded to a ⅛-in. x 1½-in. x 1½-in. structural steel angle in such a manner that when assembled on the joint the vertical leg of the angle rested against the premolded filler. A pressed steel chair, which passed under the joint, served to hold the bottom of the joint in place and support the ends of the dowel bar, maintaining it in vertical alignment. The joint was held at the top by means of pressed steel diagonal braces extending over the top of the joint and fastened to the dowel chair. Figure 19 shows details of design.

A few samples of the joint were also furnished in which an oval dowel of tubular construction, whose outside dimensions were 1 in. x 1¾ in., was sub-





FIGS. 20 AND 21. J-10 EXPANSION JOINT

stituted for the 1-in. round dowel. Though not approved for use by the Division of Highways, this joint was included in the program of tests at the University of Illinois and installed in the Armington Experimental Road.

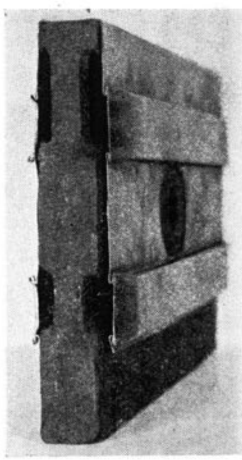
(3) J-10 EXPANSION JOINT. Figure 20 shows a short section of this expansion joint, which consisted essentially of two pieces of premolded fiber joint filler cemented to two side plates made of No. 14 gage sheet steel formed to provide space at the center of the joint for a flat steel bar, which acted as the load transmission device. The plate, which extended the full length of the joint, projected into the concrete on each side of the joint. Lugs, cut and formed from the side plates, extended outward from the joint to provide anchorage in the concrete, and the bottom of the joint was supported by means of a sheet steel base. The copper top seal was made in the form of a flanged channel set over the top of the premolded filler. The seal extended farther down on one side of the joint than on the other, placing the flange on the short side very close to the surface of the pavement. The flange on this side was short, providing very little anchorage in the concrete. No end seals were provided. Figure 21 shows details of design.

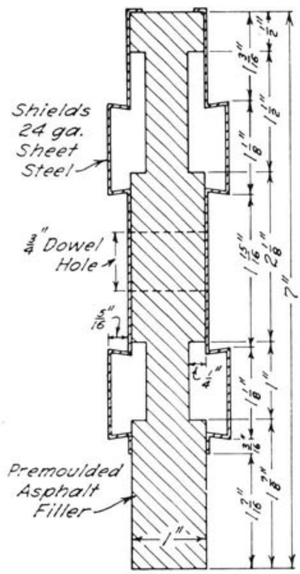
This joint was tested by the Division of Highways but was not approved for use. It was also included in the program of tests conducted by the University of Illinois and installed in the Armington Experimental Road. The Division of Highways tested joints with a ½-in. filler and plate dowels 1½ in. x 3/16 in., 1½ in. x ¼ in., and 2½ in. x 3/16 in., and a joint with a 1-in. filler and a plate 3 in. x ¼ in. Those joints included in the laboratory tests at the University had a 1½-in. x 3/16-in. dowel plate and a ½-in. filler. In the joints installed in the Armington Experimental Road, the plate was 2¼ in. x ¼ in.

The Division of Highways also tested a later design (Fig. 31) in which the dowel plate was omitted, the horizontal members of the side walls being fitted tightly together to provide load transfer.

(c) Metal-Sealed Contraction Joints

The Division of Highways made only a few preliminary tests on metal-sealed contraction joints, and no joints of this type were included in the program of tests conducted by the University of Illinois. The contraction joint was not considered less important than the expansion joint, but it was believed that if the expansion joints passed the tests the contraction joints would also, because the conditions imposed upon expansion joints installed in pavements were believed to be more severe than those imposed upon contraction joints. This belief was verified by the field investigations, which showed that the copper seals on contraction joints suffered much less than those on expansion joints.

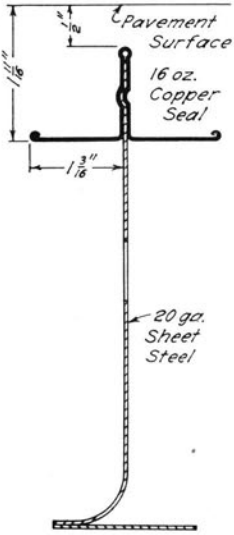


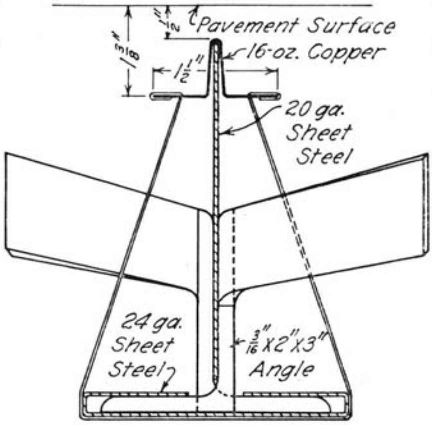


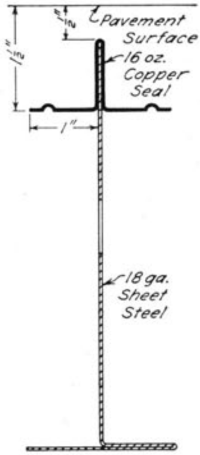
FIGS. 22 AND 23. J-11 BITUMINOUS EXPANSION JOINT

Only five contraction joints were submitted to the Division of Highways for approval; with the exception of one they all consisted essentially of a sheet steel plate cut to the form and dimensions of the cross section of the pavement, a U-shaped copper seal, and galvanized iron end closures. The copper seal was flanged to provide anchorage in the concrete and the steel plate was punched to receive the load transmission devices.

Figures 24, 25, and 26 are drawings of the J-1, J-2, and J-4 con-







FIGS. 24, 25, AND 26 (FROM LEFT TO RIGHT). RESPECTIVELY, J-1, J-2, AND J-4 CONTRACTION JOINTS

traction joints, respectively. In 1935, the J-2 contraction joint was modified to include a steel dividing plate and a copper seal similar in shape to that on the J-1 and J-4 contraction joints. This joint was furnished with the L-2 load transmission system, as was the original design. The fifth joint, J-5, was almost identical in design to the J-4 joint.

The J-1, J-4, and modified J-2 contraction joints were installed in the Armington Experimental Road in conjunction with the respective expansion joints. Four other contraction joints, all similar in design to the J-1 and J-4 joints, namely, the J-3, J-5, J-6, and J-7, also were installed in the Armington Experimental Road.

(d) Dummy Contraction Joints

This is a name commonly given to a transverse crack formed by cutting a groove in the surface of the pavement at the time the concrete is finished, or by use of a cracker strip placed on the subgrade or inserted in the concrete during the paving operations. Dummy contraction joints may also be formed by cutting a groove in the concrete with an abrasive wheel after the concrete has set. In the case where grooves are formed or cut in the surface of the pavement, the space is usually filled with a poured asphalt or other type of poured filler or a preformed filler. This type of joint is usually installed without any provisions for transferring load from one slab to the other except that which is obtained by the interlocking action of the irregular faces of the joint, commonly referred to as "aggregate interlock." However, load transmission devices are sometimes used in connection with this joint, being set at predetermined locations on the subgrade before the concrete is placed, the grooves being subsequently formed over the load transmission devices. Where mesh reinforcement is used, it is sometimes permitted to extend across the dummy joints to assure more positive aggregate interlock by holding the joint faces close together.

The dummy contraction joint had not been used by the Illinois Division of Highways at the time these studies were made. Load transmission tests were made at the University of Illinois on several variations of this type of joint, and one type was installed experimentally in the Armington Experimental Road.

(e) Open Joints with Poured Fillers

This type of joint, perhaps the first used in highway construction, has had wide usage throughout the United States. It is usually formed by placing a removable header or bulkhead at a point where a joint is desired. The space left by the header is filled with one of the many types of fillers. The joint may be installed with or without load transmission devices. Where load transmission devices are used, they are generally conventional dowels, and a special slotted header which can be withdrawn without disturbing the dowels is used. The fillers commonly used may be classified into two groups; namely, bituminous and bituminous mixtures, and rubber and rubber mixtures.

(1) Bituminous and Bituminous Mixtures. Bituminous fillers include various grades of asphalt and tar which are heated and poured into the expansion space. In Illinois the use of this type of filler, with the exception of some experimental work, has been confined principally to a blown asphalt having a minimum softening point of 180 deg. F. and a penetration from 30 to 50. This material was used almost exclusively in filling the 4-in. open joints built in pavements constructed from 1928 to 1932, inclusive, and it was also used in the two 4-in. open joints installed in the Armington Experimental Road. Throughout the United States, however, a wide variety of these materials has been used, depending somewhat upon climatic conditions.

In many states considerable use has been made of bituminous mixtures, which consist of asphalt or tar mixed with sand, lime, sawdust, cottonseed hulls, peanut shells, cotton waste, and similar materials. In many cases it would appear that the use of these materials has been based on the desire to consume the by-products of some industry or some material of which there is an overabundance in the state. Illinois has had little or no experience with the use of bituminous mixtures.

(2) Rubber Mixtures. In recent years a great deal of promotional work has been done in connection with fillers made from rubber. These generally consist of latex rubber or emulsified rubber mixed with various inert materials such as vermiculite, granulated cork, and fuller's earth. Use has also been made of mixtures of latex and bituminous materials. Some states also use mixtures of rubber and synthetic resins. Except for a slight amount of experimental work, these materials have not been used in Illinois.

(f) Preformed Joint Fillers

Preformed fillers have been widely used throughout the United States. This type of joint filler consists of an extruded or formed board which can be cut to the form and dimensions of the pavement cross section. Such fillers are made from felt and bituminous materials, vegetable fibers, cork, rubber, and wood.

(1) Bituminous Premolded Fillers. The earliest form of bituminous premolded joint fillers, which consisted of a mixture of bitumen, felt, and mineral, was not satisfactory because it was difficult to handle during hot weather. The fault was soon corrected by the addition of a layer of felt on each side of the joint. This so-called sandwich joint has had wide usage throughout the United States and is the only type of bituminous premolded joint filler that has been used in Illinois to any extent. All the bituminous premolded joints covered by the investigations reported herein were of the sandwich type. Another type which has had some use in this country is one which is similar to the bituminous and felt joint, having vegetable fibers in place of the felt.

(2) Fiber-Board Fillers. In recent years joints made from vegetable fiber boards have been widely accepted. This material, similar to the insulating boards used as sheathing in house construction, is used both plain and impregnated with an asphaltic oil. Since the beginning of 1938 the impregnated type has been used almost exclusively in pavements constructed under the direction of the Illinois Division of Highways.

(3) Miscellaneous Preformed Joint Fillers. In this category may be included boards made from granulated cork with resin or rubber as a binder, fillers formed from sponge-rubber and other rubber materials, and joints made from wood. Cork, rubber, and cork-rubber preformed joint fillers are admitted as alternates for the premolded bituminous fiber joint under the Illinois specifications, but have not been used to any extent. Preformed sponge-rubber joints were installed in the Armington Experimental Road.

Wood joints are made from various woods, the variety depending somewhat upon availability in any particular section of the country. The wood is used either untreated or, in localities which may be subject to wood-boring insects, it is treated. Wood joints have not been used in Illinois except for a few installations in the Armington Experimental Road.

A number of 1-in. open joints and fiber expansion joints in the Armington Road were fitted with a preformed rubber seal — a tubular strip of molded rubber which is molded into the bottom of the slab and driven into the top and ends of the joint opening with a mallet or hammer after the concrete has set. The seal is made wider than the joint opening so that it is compressed when inserted in the joint. The pressure of the compressed seal against the sides of the opening, intended to keep the seal from being dislodged from the joint, is further aided by longitudinal grooves in the contact surfaces of the seal which increase the mechanical friction.

(4) J-11 Non-extruding Bituminous Joint. This joint consisted of a bituminous premolded filler, supported on each side by sheet steel plates formed with two channel-shaped horizontal extrusion chambers into which the joint material may extrude when compression occurs. The thickness of the filler was reduced at the extrusion chambers to provide additional space into which the material might extrude. It was claimed that this construction would eliminate the excessive extrusion at the top which is characteristic of the ordinary bituminous premolded joint. Holes were punched through the plates and joint material to accommodate load transmission devices. A short section of this joint is shown in Fig. 22, and Fig. 23 gives the details of its construction. This joint was not tested by the Division of Highways, but it was included in the program of tests conducted by the University of Illinois and installed in the Armington Experimental Road.

(5) Sawed Joints. This type of joint is made by cutting a section of the concrete from the pavement slab by means of an abrasive cutting wheel. Experimental work on equipment for cutting joints has been conducted at the University of Illinois under the direction of Professors Huntington and Crandell. Their work is discussed in detail later in this bulletin (see page 244).

8. *Load Transmission Devices.* — Many highway engineers have long been of the opinion that some means had to be provided for strengthening the transverse edges formed by expansion or contraction joints, and for reducing the relative deflections which occurred when heavy wheel loads passed from one slab to another separated by a free joint. Many different kinds and types of load transmission devices have been developed. The devices submitted for approval in Illinois, those tested by the University of Illinois, and those installed in the Armington Experimental Road are described below:

(a) Plain Dowel

The plain dowel consists of a round steel dowel, usually ¾ in. in diameter and 24 in. long, spaced at various intervals across the pavement, but usually 12 to 15 in. center to center. The dowel bar extends across the joint and is embedded in the concrete on both sides. One end

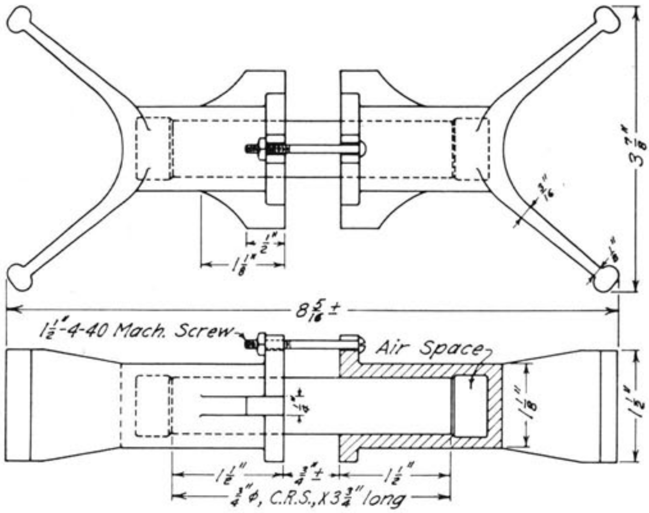


FIG. 27. L-1 LOAD TRANSMISSION DEVICE

of the bar is coated with grease and provided with a short sheet metal expansion sleeve to break the bond between the bar and the concrete and to provide room for movement at the end of the bar, thus permitting the joint to open and close without excessive stresses being set up in the concrete. The dowel bars used in Illinois, ¾ in. in diameter and 24 in. long, are held in position and proper alignment by means of metal supports driven in the subgrade and by ½-in. round deformed spacer bars wired to the dowels. The use of special installation devices which have been developed by various manufacturers is also permitted. The ¾-in. dowel bar, 24 in. in length, was included in the program of tests at the University of Illinois, and was also used in conjunction with several joints installed in the Armington Experimental Road.

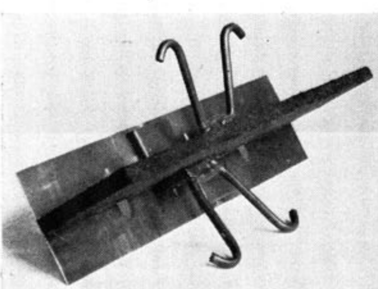
(b) L-1 Load Transmission Device

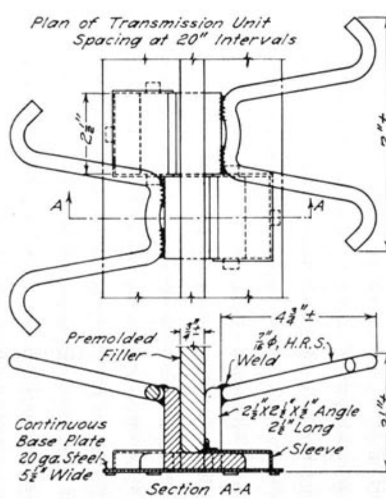
This device, commonly referred to in Illinois as the wing anchor load transmission device, consisted essentially of a short round steel dowel enclosed in sockets or sleeves made from malleable cast iron. The name "wing anchor" was derived from the wings cast on the end of the sockets to provide anchorage in the concrete. The sockets were machined to fit the dowel within a close tolerance. Figure 2 is a photograph showing a wing anchor device assembled on a J-1 expansion joint, and Fig. 27 is a drawing of the unit. This device, approved for use in Illinois in 1935, was included in the program of tests at the University of Illinois and used in conjunction with several joints installed in the Armington Experimental Road.

(c) L-2 Load Transmission Device

The L-2 load transmission device consisted essentially of sections of structural steel angle approximately 12 in. long and a sheet steel base sleeve. Each section of angle had three lugs cut and formed from its vertical leg so that they extended perpendicularly from that leg to provide anchorage in the concrete. The angles were placed end to end in the base sleeve with the horizontal legs extending alternately in opposite directions. Either an air-chamber expansion joint, a premolded expansion joint, or a metal contraction joint can be placed between the vertical legs of the angles, thus making the load transmission feature a more or less integral part of the joint. When installed in a pavement, alternate angles are anchored in the concrete on opposite sides of the joint and the horizontal leg of each angle extends across the joint, supporting the edge of the opposite slab.

The L-2 device was approved for use in Illinois in July, 1934. Its use in conjunction with J-2 joints was suspended in 1935 when approval of this joint was withdrawn. Later it was approved for use with the preformed joints adopted in 1938. Figure 4 shows a short section of a J-2 expansion joint with the L-2 device, while Fig. 5 shows the details of design of the J-2 expansion joint. The L-2 load transmission device was included in the program of tests at the University of Illinois and installed in the Armington Experimental Road.





FIGS. 28 AND 29.
L-3 LOAD TRANSMISSION DEVICE

(d) L-3 Load Transmission Device

This load transmission device (Figs. 28 and 29) consisted of a series of modified L-2 angles supported on a sheet steel base plate extending the full length of the joint. Each unit consisted of two $2\frac{1}{2}$-in. x $2\frac{1}{2}$-in. x $\frac{1}{2}$-in. structural steel angles, $2\frac{1}{2}$ in. in length, placed end to end, but on opposite sides of the joint in such a manner that the horizontal leg of each angle passed through a recess cut in the joint filler next to the base plate, thus spanning the joint formed by the filler. The vertical leg of each angle was provided with a two-pronged, hot-rolled steel rod welded thereto for anchoring the unit in the concrete. The horizontal leg of each angle was enclosed in a sheet steel sleeve, fastened to the base plate, which provided the expansion space needed when the joint closed. This device was submitted for approval in connection with the use of preformed joints, but was not approved for use in Illinois.

(e) L-4 Load Transmission Device

This device consisted essentially of a short, cold-rolled steel dowel $\frac{3}{4}$ in. in diameter and $4\frac{1}{2}$ in. long, the ends of which were enclosed in sockets made from gray cast iron, machined to fit the dowels to a close tolerance. A particular feature of this unit was the leg cast integrally with the socket, which extended outward from the joint and downward to the subgrade to serve as a support for the joint with which the device was used. Figure 12 shows a J-6 expansion joint with this load transmission device in place, and Fig. 13 shows the J-6 expansion joint equipped with the L-4 load transmission device. This device, tested by the Illinois Division of Highways but not approved for use, was included in the program of tests conducted by the University of Illinois and installed in the Armington Experimental Road.

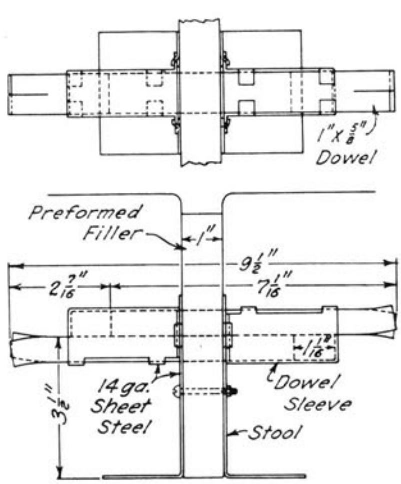
(f) L-5 Load Transmission Device

This load transmission device consisted of two rectangular steel dowels, 1 in. x $\frac{5}{8}$ in., overlapping each other and inserted into two formed sheet steel sockets, one dowel being attached to each socket in such a manner that expansion and contraction of the pavement caused each dowel to slide against the other, in and out of the socket to which the other was attached. Each socket was attached to a sheet steel stool of sufficient height to hold the dowels at the proper distance above the subgrade. The joint or joint filler was inserted between and fastened to the sheet metal stools, the dowels passing through a hole in the joint or joint filler. Figure 6 is a photograph showing the device assembled on a metal joint, and Fig. 30 is a drawing of it. This device, tested by the Illinois Division of Highways but not approved for use, was included in the program of tests at the University of Illinois and installed in the Armington Experimental Road.

(g) L-6 Load Transmission Device

This device, an integral part of the J-10 expansion joint, consisted of a flat steel bar placed in a horizontal plane at the mid-depth of the slab so that it spanned the joint and extended into the concrete on each side. The bar, extending the full length of the joint, was enclosed in sleeves formed from No. 14 gage steel which also served as side walls for the expansion joint. Space was provided between the ends of the sleeves and the edges of the bar to permit movement of the slab during periods of expansion of the concrete. Lugs were punched out of the metal side plates at intervals to anchor the side plates to the concrete. A J-10 expansion joint with the L-6 load transmission device is shown in Fig. 20, while Fig. 21 shows a cross section of the joint. The plate dowel in the joints tested by the Division of Highways was of various sizes; in those tested in the laboratory at the University of Illinois it was 1½ in. x $\frac{3}{16}$ in. The joints installed in the Armington Experimental Road had a plate dowel 2½ in. x ¼ in.

The Division of Highways also tested a later design (Fig. 31) in



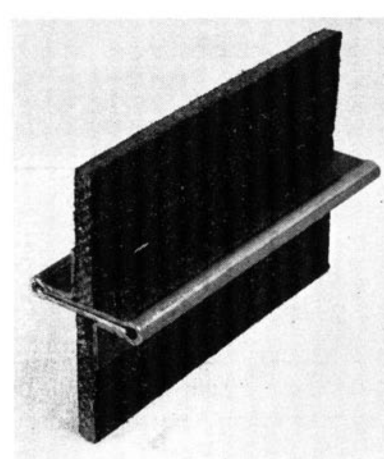
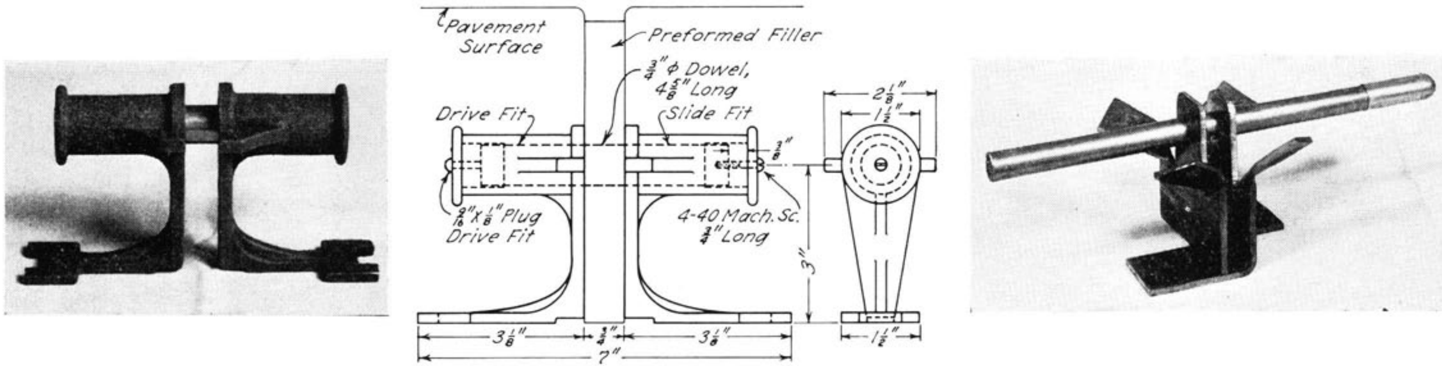


FIG. 30 (AT LEFT). L-5 LOAD TRANSMISSION DEVICE
FIG. 31 (AT RIGHT). MODIFIED J-10 EXPANSION JOINT



FIGS. 32, 33, AND 34 (FROM LEFT TO RIGHT). RESPECTIVELY, L-7 LOAD TRANSMISSION DEVICE; THE SAME DEVICE SKETCHED; L-8 LOAD TRANSMISSION DEVICE which the plate dowel was omitted, the horizontal members of the side walls being fitted tightly together to provide load transfer.

(h) L-7 Load Transmission Device

This device consisted of a $\frac{3}{4}$-in. cold-finished, round dowel, $4\frac{5}{8}$ in. long, the ends of which were enclosed in malleable cast iron sockets or sleeves. Each sleeve was $2\frac{1}{2}$ in. in overall length, providing space beyond the end of the dowel to permit movement during periods of expansion of the concrete. At the extreme end each sleeve was provided with a flange which served to anchor the device in the concrete. The end of the sleeve next to the joint was provided with a flange having projections $\frac{1}{4}$ in. thick, extending horizontally to each side for a distance of $\frac{5}{16}$ in., the purpose of which was to provide increased bearing of the sleeve on the concrete next to the joint. The lower side of the flange was carried down to the bottom of the joint and terminated in a foot which extended perpendicularly from the joint and horizontally along the subgrade.

The design of the L-7 device is shown in Figs. 32 and 33. Tested by the Illinois Division of Highways but not approved for use, it was not included in the program of tests conducted by the University of Illinois but was installed in one joint in the Armington Experimental Road.

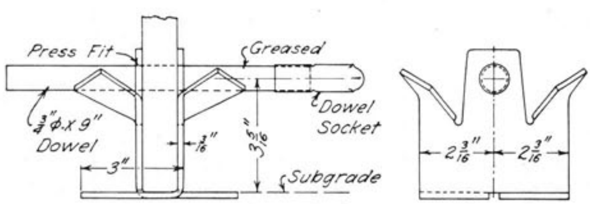


FIG. 35. L-8 LOAD TRANSMISSION DEVICE

(i) L-8 Load Transmission Device

This device (Figs. 34 and 35) consisted of a $\frac{3}{4}$-in. cold-finished, round dowel 9 in. in length, bonded in the concrete on one side of the joint and greased and provided with an expansion socket on the other side. At each of the faces of the joint the bar passed through a $\frac{3}{16}$-in. steel plate, the bar being welded or otherwise securely fastened to one plate and free to move in the other. The plates were bent in the form of angles and so arranged that the horizontal leg of each angle passed under the joint filler and extended under the adjacent concrete slab, one half of the leg being removed from each angle to permit this. Two lugs cut from the vertical leg of each angle and bent outward served to anchor the device in the concrete. Sheet steel caps were placed on the ends of the horizontal legs to provide for movement of the slab during periods of expansion.

The L-8 device was tested by the Division of Highways but was not approved for use. It was not included in the program of tests at the University of Illinois nor installed in the Armington Experimental Road.

(j) L-9 Load Transmission Device

This device consisted of a cold-finished, round steel bar, a sleeve made from galvanized steel, and two flanged stress reducers formed from mild steel plate. The steel bar was ¾ in. in diameter and 6 in. long. The sleeve, made from No. 22 gage sheet steel and rolled tightly around the bar, was 7½ in. long and had metal thimbles or stops soldered in its ends. Small indentations in the sleeve held the bar centered so that there was approximately ½ in. of free space between each end of the bar and the stops. The sleeve was weakened at the center by five slots cut out of the metal. The stress reducers were made from ⅛-in. steel plate in two halves riveted together. When the halves were assembled they formed a tube 2 in. long with diametrically opposed flanges extending outward $^9/_{16}$ in., whose inside diameter was such that it fitted tightly over the sleeve enclosed bar. It was the purpose of the reducer to increase the effective area through which the pressure imposed on the bar by loads was distributed to the concrete. When completely assembled the two stress reducers were centrally located on the bar, one on each side and in contact with the expansion joint or expansion joint material. Sheet steel chairs were provided to help support the joint and hold the bar in alignment.

The design of the L-9 device is shown in Figs. 36 and 37. Tested by the Illinois Division of Highways but not approved for use, it was not included in the program of tests conducted by the University of Illinois nor installed in the Armington Experimental Road.

(k) L-10 Load Transmission Device

The L-10 load transmission device consisted of a ¾-in. cold-finished, round steel dowel 5 in. in length, enclosed in sleeves or sockets of welded steel construction. Each sleeve was provided with a flange adjacent to the joint to increase the bearing on the concrete,

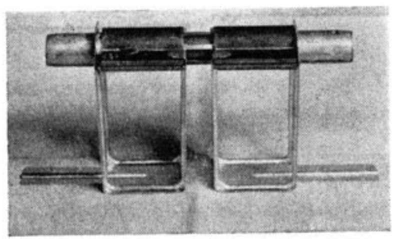


Fig. 36. L-9 Load Transmission Device

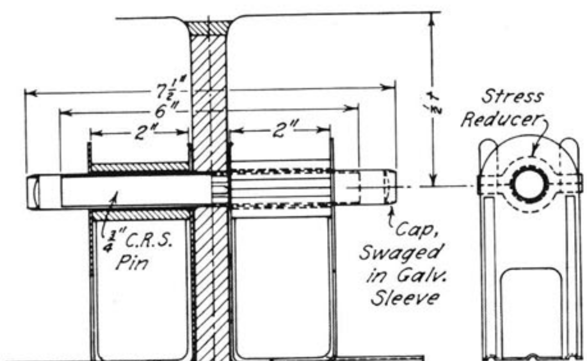


Fig. 37. L-9 Load Transmission Device

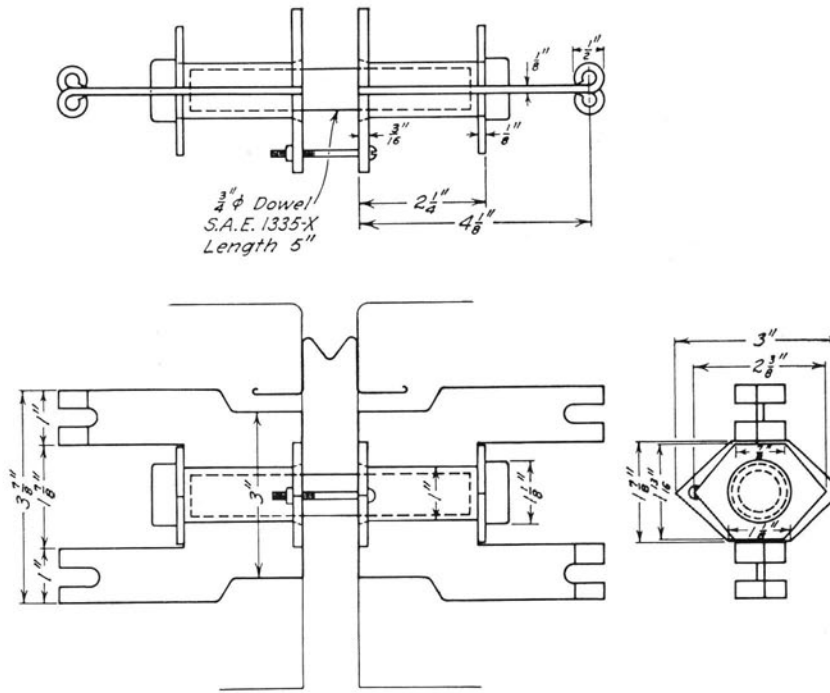
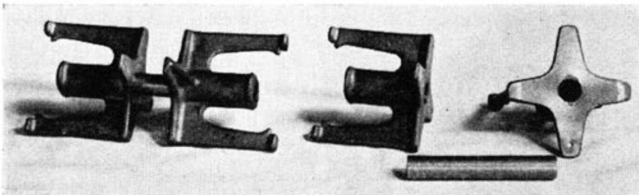
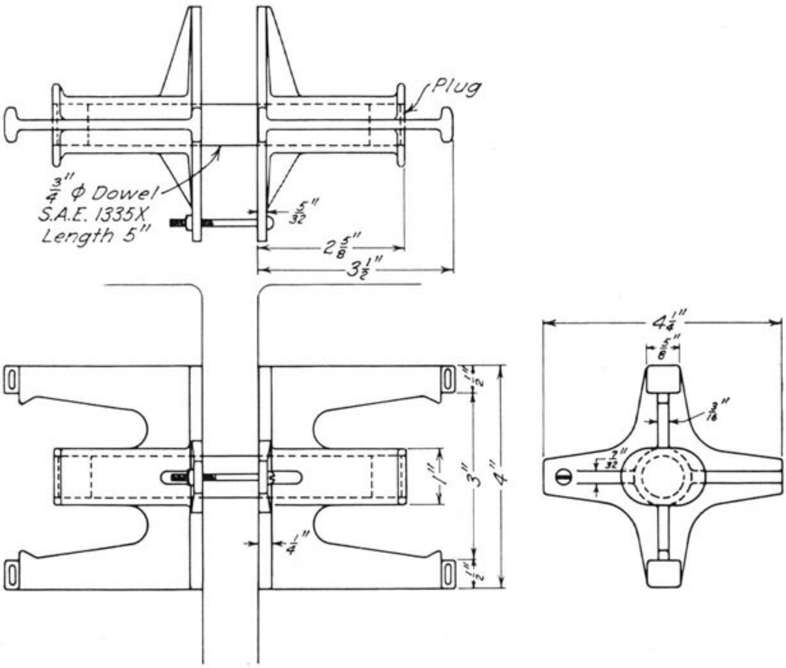


Fig. 38. L-10 Load Transmission Device





FIGS. 39 AND 40. L-11 LOAD TRANSMISSION DEVICE

and with projections extending from the flange above and below the sleeve to provide anchorage in the concrete. Space was provided between the dowel and the end of each sleeve to permit movement of the concrete during periods of expansion.

Figure 14 shows L-10 load transmission devices assembled on a metal expansion joint, while Fig. 38 is a drawing of the device. It was tested by the Division of Highways but not approved for use, and was included in the program of tests conducted by the University of Illinois and used in conjunction with the J-7 joint installed in the Armington Experimental Road.

(l) L-11 Load Transmission Device

This device, shown in Figs. 39 and 40, was similar in all respects to the L-10 device except that the sleeves or sockets were made from malleable cast iron. It was tested by the Illinois Division of Highways but not approved for use, was included in the program of tests conducted by the University of Illinois, and was used in conjunction with several joints installed in the Armington Experimental Road.

(m) L-12 Load Transmission Device

This device was very similar to the standard wing anchor device, L-1. Perhaps the greatest difference between the two was that the sockets of the L-12 were cored all the way through and the opening at the end closed with a plug, whereas the sockets for the standard

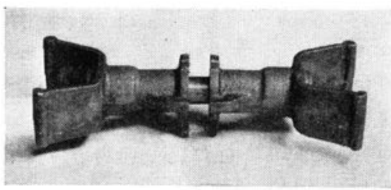


FIG. 41. L-12 LOAD TRANSMISSION DEVICE

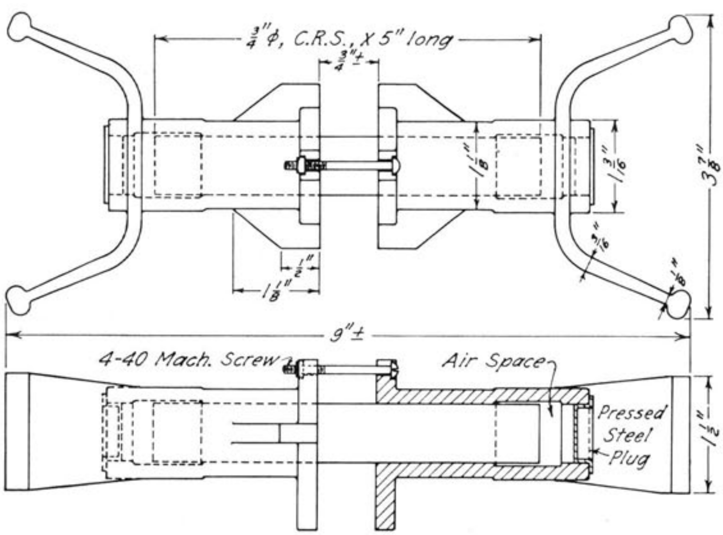


FIG. 42. L-12 LOAD TRANSMISSION DEVICE

wing anchor device were made in one piece. There were some minor differences between the two devices in the shape of the sockets and wings which, however, would not be expected to affect the load transmission properties substantially. The design of the L-12 device is shown in Figs. 41 and 42. It was tested by the Illinois Division of Highways, but was not included in the series of tests conducted by the University of Illinois nor installed in the Armington Experimental Road.

(n) L-13 Load Transmission Device

This load transmission device was an integral feature of the J-9 expansion joint described on page 36. The L-13 device was not tested by the Illinois Division of Highways, but it was included in the program of tests conducted by the University of Illinois and was installed in the Armington Experimental Road.

The 1-in. round dowel was understood to be the standard design, but the manufacturer also submitted to the University samples equipped with oval dowels made from steel tubing with 0.15-in. wall thickness whose outside dimensions were 1 in. and 1¾ in. The dowels were available in different lengths. The specimens tested at the University included dowels 17 in., 19 in., and 21 in. long.

(o) L-14 Load Transmission Device

Figure 43 shows a joint in place on the subgrade, while Fig. 44 is a modified design of this load transmission system. In this design, ⅜-in. round rods support the dowels instead of the flat plate and angle described on page 54. This load transmission assembly was not tested by the Illinois Division of Highways nor by the University of

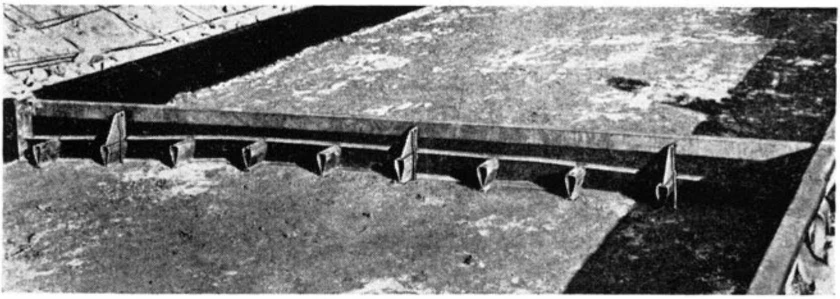


FIG. 43. L-14 DOWEL ASSEMBLY

Illinois. It was installed in the Armington Experimental Road in conjunction with premolded fiber expansion joints.

This load transmission system was an assembled unit consisting of pressed steel dowels held in proper position and alignment by spacer members welded to the dowels. The dowel, of tubular construction,

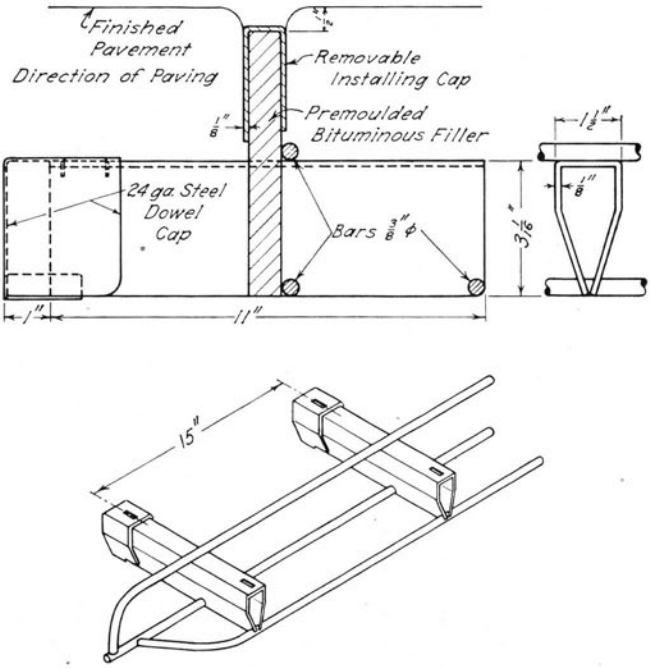


FIG. 44. L-14 DOWEL ASSEMBLY

essentially triangular in cross section, was made from ⅛-in. sheet steel. The dowels were 10 in. long, welded at regular intervals to a flat steel base plate which extended the full width of the pavement. The dowels were further supported by means of a small angle welded to their tops and extending the full length of the joint. This angle served also to support the joint filler which was notched to fit the dowels. One end of each dowel was provided with a pressed steel cap to allow for movement during periods of expansion of the concrete.

(p) L-15 Load Transmission Device

This device consisted of a No. 14 U. S. S. gage black steel plate extending the full width of the pavement and anchored in the concrete on each side of the joint by means of lugs pressed out of the plate. One row of lugs was located near the top of the plate, extending downward at an angle of 45 deg. Another row, located near the bottom of the plate, extended outward from the other side of the plate and upward at the same angle. The lugs were flanged over at the ends to provide better anchorage in the concrete. A ⅜-in. hot-rolled, triangular bar was spot welded to each row of lugs at the junction of the side plate and lugs. Movements at the joint are taken care of by means of flexure in the vertical plate and bending of the lugs. When used as an expansion joint, pieces of fiber joint material ¼ in. thick are cemented to each side of the plate.

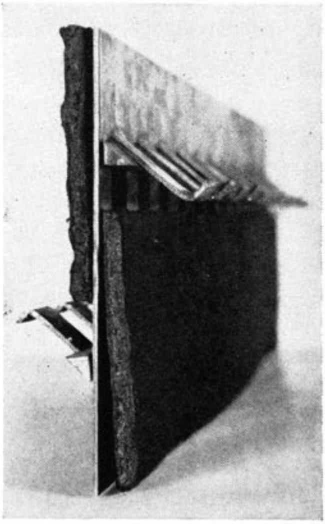


FIG. 45. L-15 LOAD TRANSMISSION SYSTEM

This L-15 device provides for very limited movements and is intended to be used at rather close intervals of from 12 to 15 ft. Shear is transmitted through the lugs which anchor the plate into the concrete. A short section of this device arranged as an expansion joint is shown in Fig. 45; Fig. 46 is a drawing of the device. Not tested by the Illinois Division of Highways, the L-15 was included in the program of tests conducted by the University of Illinois but was not installed in the Armington Experimental Road.

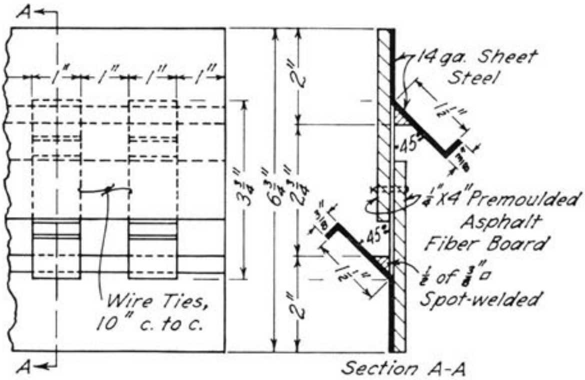


FIG. 46. L-15 LOAD TRANSMISSION SYSTEM

(q) L-16 Load Transmission Device

This dowel, commonly known as the spade dowel, was a malleable iron casting having a circular projection which extended into the concrete on one side of the joint and a triangular shaped spade and shoe which extended downward to the bottom of the slab, across the joint, and under the slab on the other side. These units were used in pairs, being placed side by side, one on each side of the joint. Through this arrangement a load on either side of the joint is partially transmitted to the adjacent slab through the shoe that extends under the loaded slab. This device may be used in conjunction with one of the various types of preformed joint filler.

Figure 47 shows a pair of L-16 dowels and Fig. 48 gives the details of design. The L-16 device was not tested by the Illinois Division of Highways. It was included in the program of tests conducted by the University of Illinois, but was not installed in the Armington Experimental Road.

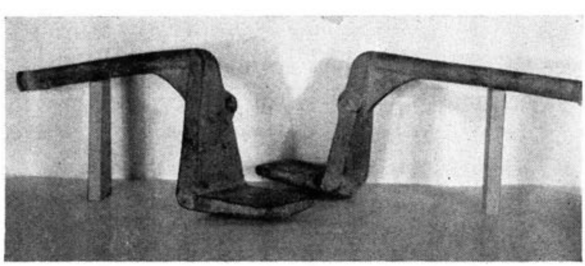


FIG. 47. L-16 LOAD TRANSMISSION DEVICE

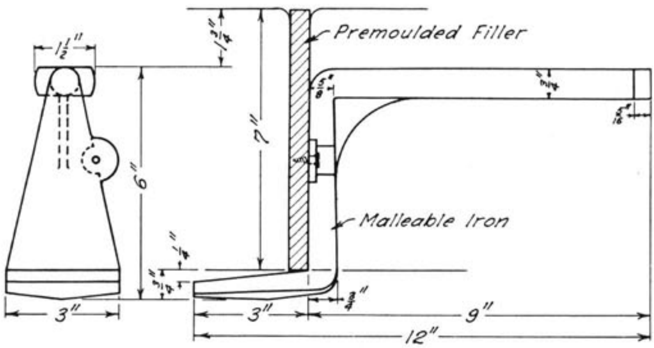


FIG. 48. L-16 LOAD TRANSMISSION DEVICE

(r) L-17 Load Transmission Device

This device consisted of two identical pieces made from $3/16$-in. low carbon steel plate cut and bent to form a horizontal supporting foot, a vertical wall which extended from the bottom of the slab up the face of the joint to the mid-depth of the slab, and a flat dowel which extended horizontally into the concrete (Figs. 49 and 50). The supporting foot was $2\frac{1}{4}$ in. wide and $2\frac{13}{16}$ in. long. The width of the vertical wall was $4\frac{1}{2}$ in. and its height varied with the thickness of the pavement in which it was used; in the units submitted for these

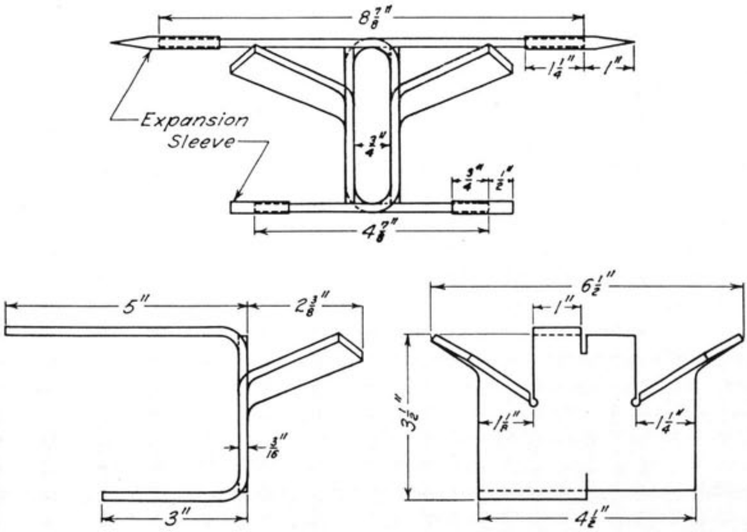


FIG. 49. L-17 LOAD TRANSMISSION DEVICE

tests, this dimension was approximately 3½ in., the device being designed for a 7-in. pavement. The flat dowel was $\frac{3}{16}$ in. thick, 1 in. wide, and $4\frac{13}{16}$ in. long. Lugs approximately 1 in. wide and 2¼ in. long, cut from the vertical wall to provide anchorage in the concrete, were bent out so as to extend in the opposite direction from the supporting foot and dowel.

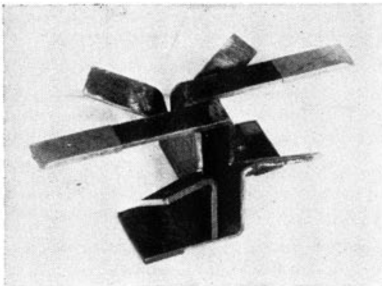


FIG. 50. L-17 LOAD TRANSMISSION DEVICE

The component pieces of the device were placed one on each side of the joint filler so that the lugs on each piece extended into the concrete on that side of the joint, the supporting foot passing across the joint and under the opposite slab, and the dowel passing through the filler and into the concrete of the opposite slab. The foot and dowel on each piece were cut so that when the device was assembled they lay side by side with just enough clearance to prevent binding. When used with a ¾-in. joint, the supporting feet extended under the slab a distance of 2 in., and the length of dowel embedment was about 4 in. Folded sheet steel sleeves were placed over the ends of the supporting feet and the dowels to provide for the movement which takes place when the concrete expands. The dowels were greased to break the bond and reduce resistance to movement.

III. Laboratory Tests

9. *University of Illinois Tests.* — In May, 1937, a committee from the engineering staff of the University of Illinois was requested by the governor of Illinois to make an investigation and report on the status of expansion joints used and proposed for use in state highways. As a part of this investigation a series of laboratory tests was planned and meetings were held with representatives of the manufacturers of joints and joint materials, who later furnished samples of their respective types of expansion joints and materials and requested their inclusion in the tests. Some of the joints submitted had never been installed in Illinois pavements, while others were in use under the specifications of the Division of Highways then current. Tests on 12 types of joints were made during the summer of 1937; later three more joints were submitted which were tested late in 1937.

(a) Object of Tests and Description of Joints Tested

Laboratory tests are useful when they indicate the properties of material and its probable behavior under service conditions. It is evident that the primary object of the use of expansion joints is to provide for expansion and contraction of the concrete pavement in order to avoid blowups due to compressive forces and transverse cracking, with the resulting faulting of adjacent sections and general roughness of the road surface.

The joints tested included both the air-chamber and the preformed filler types. Both were intended to allow space for the expansion of the concrete due to temperature and moisture changes, and for the gradual growth of the pavement due to the opening and incomplete closing of transverse cracks. The air-chamber joint consisted of a sheet steel box offering negligible resistance to closing, while the preformed filler offered increasing resistance to closing, withstood compression to about half its original thickness, and exhibited a partial recovery in thickness when pressure was removed.

A feature of most of the joints was a load transmission device, which transmitted shears due to traffic loads across the joint. An effective load transmission device should not only produce a smooth alignment of pavement surfaces at the joint, but also reduce the stresses in the pavement due to the applied loads.

Fourteen joints or partial joint assemblies, including ten different load transmission devices, were tested. The joints and load transmission devices are described in detail in Chapter II, Sections 7 and 8, pages 29–58.

(b) Outline of Laboratory Tests

When the laboratory tests were planned, it appeared that information was desirable on (a) the ability of the joints to permit frequent opening and closing; (b) the effectiveness and durability of seals used to exclude dirt, sand, gravel, and foreign matter which might prevent the effective closure of the joint; (c) the ease of installation of such joints and their effect upon the quality of the concrete placed around and against the joint; and (d) the effectiveness of the load transmission devices which were submitted with most of the joints.

It is likely that laboratory test specimens will always be made under more favorable conditions of fabrication, molding, and curing than actual pavements; furthermore, the tests will not duplicate the varied and destructive actions of traffic and weather found in service. At best, the laboratory tests can determine only the relative behavior of a group of joints under conditions that can be reproduced with a fair degree of uniformity.

The principal objectives of the tests conducted during these investigations were (a) the determination of the strength and stiffness of the load transmission device; (b) a study of the effectiveness of anchorage of the copper seal; (c) a study of the resistance of the copper seal to fatigue, when the joint was opened and closed a large number of times; (d) a study of the stability and tightness of the joint when subjected to vibration and pressure from the fresh concrete as it was placed and compacted in the form; and (e) tests of the compressibility of the various preformed fillers used in joints, as well as of the tendency for localized compressive failure of the concrete surrounding the joint when the joint was closed and subjected to compressive forces.

(c) Materials and Making of Test Specimens

Nearly all of the tests made required the embedment of lengths of the expansion joint in a section of concrete slab, in order that the joint might be subjected to various loads and displacements comparable to those occurring in service. The concrete mixtures used in the various tests are indicated in Table 3, which also lists the compressive strengths of control cylinders made for the respective test specimens.

The opening-closing tests and collapsing studies of joints were made at the Springfield laboratory of the Illinois Division of Highways because special equipment was available there for that purpose. The materials used there were those in general use in the laboratory for Class A highway concrete. The materials used at the University were a standard portland cement, Wabash River torpedo sand, and 1¼-in.

TABLE 3
COMPRESSIVE STRENGTH OF CONCRETE USED IN TEST SPECIMENS[1]
(University of Illinois Tests)

Load Transfer Test			Copper Seal Pull-out Test		Opening-Closing Test		Compression Test	
Ref. symbol	Number of cyls.	Strength at 14 days, lb. per sq. in.	Number of cyls.	Strength at 14 days, lb. per sq. in.	Number of cyls.	Strength at 7 days, lb. per sq. in.	Number of cyls.	Strength at 14 days, lb. per sq. in.
TESTS OF COMPLETE JOINTS								
J-1	9	4,050	6	4,140	6	4,380	2	4,180
J-4	12	3,980	9	3,820	6	4,320	4	4,000
J-5	10	4,110	8	3,890	6	4,350	2	4,030
J-6	10	4,070	8	4,270	6	4,110	3	4,280
J-7	13	4,140	3	4,410	6	4,230	6	3,920
J-2	13	4,040	9	3,690	6	4,530	4	4,000
J-3	14	4,080	3	3,830	6	5,040	6	4,100
J-10	10	4,130	7	3,920	6	4,800	5	4,090
J-8	10	4,410	7	4,230	6	4,470	5	4,010
J-9	14	3,910	6	3,830	6	4,190	5	4,000
Average		4,090		4,000		4,440		4,090
MISCELLANEOUS TESTS								
Oval pipe	9	4,190						
L-11	18	4,050						
No load transmission device	7	4,580						
No load transmission device	6	4,070						
Wire mesh	6	3,790						
L-15 (tens.)	3	3,510					3	3,465
L-15 (comp.)	3	3,550						
L-16	3	3,660						
J-11							3	3,845
DATA REGARDING MIXTURES								
Proportioning by weight	1:3:5		1:3:5		1:1.9:3.1		1:3:5	
Average slump, in.	2.0		2.0		1.9		2.0	

[1] Compressive strengths from tests of standard 6 in. x 12 in. cylinders.

gravel. In all cases, the concrete was batch-mixed, placed in wooden forms and compacted and worked into contact with the copper seals, dowels, anchors, and other projecting parts by spading and vibration. A surface vibrator was used at the Springfield laboratory, an internal vibrator at the University.

After the slabs were cast, they were cured under wet burlap (the 14-day slabs for seven days, the 7-day slabs for six days); then the

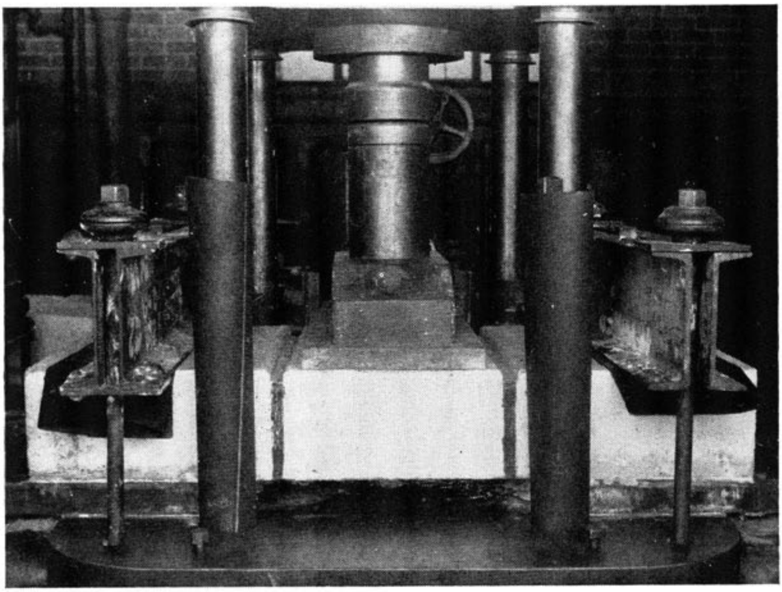


FIG. 51. SLAB AND TESTING APPARATUS, LOAD TRANSFER TESTS.
(a) METHOD OF SUPPORTING AND LOADING THE SLAB

burlap was removed and the slabs were allowed to dry in the air of the laboratory until they were tested.

(d) Load Transmission Test

(1) TEST ARRANGEMENT AND PROCEDURE. The load transmission test was planned to study the effectiveness of dowels or other units in transferring load across an expansion joint. For this purpose the test specimen consisted of three slabs 7 in. thick, 2 ft. wide and about 16 in. long, connected by two expansion joints shown in Fig. 51a. The manufacturers were requested to submit joints ¾ in. wide, but in some cases other widths were substituted. The test specimen was placed in the testing machine with the two end slabs uniformly supported and firmly anchored to the heavy cast steel table of the machine. To produce uniform distribution, load was applied to the middle slab through a spherical bearing block and heavy distributing plates. The arrangement was such as to minimize bending moment across the joint.

Deflections across each joint were measured at two sections, 13½ in. apart and above the dowels, by means of Ames 0.001-in. micrometer dials attached to a bridge which was supported independently of the slab. The deflection was taken as the difference in vertical movement at opposite sides of the joint. Figure 51b shows the test setup, including the dial arrangement.

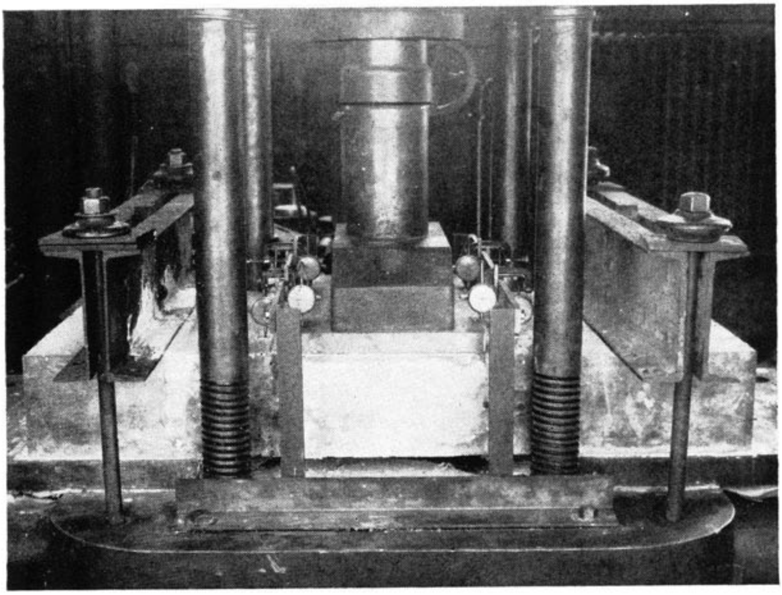


FIG. 51. SLAB AND TESTING APPARATUS, LOAD TRANSFER TESTS.
(b) SHOWING DIAL SET-UP FOR MEASURING DEFLECTIONS

In general, five specimens were made for each type of joint or load transmission device tested. In testing two of these specimens, starting with the deflection dials set at zero, loads of increasing magnitudes were applied, beginning with 2,000 lb. and increasing by 2,000-lb. increments. Each load was applied and released three times before the specimen was subjected to the next higher load. Careful measurements were made each time a load was applied and released, in order to determine the deflection under load and the residual deflection or permanent set after the load was removed. The averages of the three readings of deflection and permanent set were used in plotting load deflection and permanent set curves for the joint or device. In testing the other three specimens, the load was applied in 1,000-lb. increments, beginning with 1,000 lb. The loads were applied and released only once before the next higher one was applied. It is believed that this method gave results equally as reliable as the method used to test the first specimens.

Eight specimens containing the L-15 load transmission device were included in this program. In four of the specimens, the joints were arranged so that the load produced tension in the load transmission plate of the joint; in the remaining four specimens, so that compression was produced in this member. Before applying the load, each joint was opened ⅛ in. to break the bond and produce conditions similar to those which would prevail in service.

TABLE 4
MAXIMUM LOADS CARRIED BY LOAD TRANSMISSION TEST SPECIMENS[1]
(University of Illinois Tests)

Kind of Load Transmission Device	Joint Width	Load, in lb., on Test Specimen No.					Average Load
	in.	1	2	3	4	5	lb.
L-1	¾	48,000	47,000	48,800	51,200	49,100	48,820
¾-in. dowel, 24 in. lg.	¾	28,000	29,400	27,300	27,300	28,450	28,090
L-1	¾	49,300	49,900	47,000	49,700	47,600	48,700
L-4	½	46,700	48,700	52,500	49,200	54,800	50,380
L-10	1	42,200	46,980	40,500	43,160	43,050	43,180
L-2	1	16,000	15,600	16,000	17,200	17,000	16,360
L-5	1	36,150	34,320	35,160	33,670	33,200	34,500
L-6 (3/16 in. x 1½-in. plate)	½	22,500	22,000	20,500[2]	22,500[2]	22,000	21,900
¾-in. dowel, 24 in. lg.	¾	31,600	30,200	28,300	30,000	33,350	30,690
1-in. dowel, 17 in. lg.	¾	47,500	53,300	50,000	51,600	56,200	51,720
Oval pipe	¾	53,740	50,250	48,900	......	......	50,960
L-11	¾	76,500[3]	77,000[3]	32,000	42,600	34,000	36,200[3]
L-15 (tension)	⅜	24,000	18,500	27,900	23,400	......	23,450
L-15 (comp.)	⅜	13,300	15,000	21,400	18,600	......	17,075
L-16	¾	67,200	66,400	67,100	......	......	66,900
None	(4)	85,550	93,100	......	......	......	89,320
None	(4)	95,700	46,600	69,700	......	......	70,670
Wire mesh	(4)	110,700	128,300	106,400	......	......	115,130

[1] Where load transmission devices were of the dowel type, there were four in each specimen.
[2] Load increased beyond this due to wedging of middle slab against end slabs, but this is considered the maximum load under normal conditions.
[3] Omitted from average (see detailed notes regarding test).
[4] Narrow transverse crack.

Tests were made on three specimens containing the L-16 load transmission device, which can transmit shear in one direction only; in practice it is used in pairs. In these tests, however, the four dowels in each specimen were all placed so as to develop their full effectiveness.

(2) DATA FROM LOAD TRANSMISSION TESTS. A summary of the maximum loads carried by the various types of joints is given in Table 4. It will be noted that the width across the joints varied, a factor which undoubtedly had a very great effect on the load carried. Thus, if the bending strength of the dowel governed the failure, the strength of the joint should have varied inversely as the width of opening spanned by the dowel. Several of the joints did not meet the Illinois specifications, which prescribed air-chamber joints and a clear opening in the joint of ¾ in.

A detailed statement as to the manner of failure of each slab and sketches showing the location of cracks are on file with the original test data. Typical failures of some of the slabs are shown in Fig. 52. A few notes regarding the manner of failure of the test slabs are given below. The slabs are identified by the type of load transmission device used.

Conventional Dowel with J-4 Expansion Joint. These joints were equipped with conventional ¾-in. round dowels, 24 in. long. The specimens failed by diagonal fractures of the upper corners of the middle slab next to the joint and above the copper seal, and the lower corners of the outer slabs. Bending of dowels and crushing of concrete were also noted.

FIG. 52. JOINTS AFTER FAILURE IN LOAD TRANSFER TESTS (UNIVERSITY OF ILLINOIS TESTS)

Conventional Dowel with J-8 Expansion Joint. This joint was equipped with ¾-in. round dowels, 24 in. long. In the tests of these specimens there was little cracking until the maximum load was approached. Generally, failure was due to bending of the dowels and crushing and breaking of concrete beneath the dowels in the end slabs.

L-1 with J-1 Expansion Joint. There were few cracks in these slabs until the maximum load was approached. The specimens failed generally through horizontal splitting of an end slab and cracking through to the top because of the prying effect of the load transmission device. Frequently a corner or portion of the slab adjoining the joint was split off.

L-1 with J-5 Expansion Joint. The failure of these slabs was similar to that of the L-1 with J-1 expansion joint slabs. Cracking usually started in the plane of the load transmission devices or the copper seal. General cracking followed as failure was approached, principally in the region of load transmission devices, which in some cases seemed to split the slab in a horizontal plane.

L-2 with J-2 Expansion Joint. This joint had shelf angles 12 in. long, with an outstanding leg of the angle projecting about 1¼ in. alternately to one side and the other of the joint. Thus in a 24-in. length of joint, only 12 in. of shelf angle was effective in a given direction. The joints used in the first two test slabs had the projecting angles diagonally opposite in the center slab. In the last three specimens the 12-in. angle supporting the center slab was placed on the centerline of the slab, and two 6-in. sections with outstanding legs under the end sections were placed at the sides, it being thought that with a symmetrical support the center section would be less likely to rotate, and the test might yield more accurate results. This arrangement, however, did not seem to produce results particularly different from the original one.

In placing the slab in the testing machine, there seemed to be considerable play between the slab and the shelf angle. As the load approached the maximum, there was evidence of some twisting of the slab and of large deflection of the supporting flange of the angle. At failure the slab cracked through vertically behind the anchor lugs holding the angles.

L-4 with J-6 Expansion Joint. The first cracks in the slabs occurred at about 90 per cent of the maximum load, or roughly from 43,000 to 46,000 lb. Cracking started near the copper seal or in the plane of the load transmission units, which were gray iron castings encasing ¾-in. steel dowel pins. At failure, in some cases, the gray iron casting was broken in several pieces. The air chamber in this joint was only ½ in. wide; had it been ¾ in., it is doubtful, in view of the failure of castings noted above, that such high loads would have been reached.

L-5 with J-3 Expansion Joint. Few cracks developed in the test specimens until the maximum load was approached. The cracks generally ran from the dowel to the bottom of the supporting slab, breaking off the supporting corner of the end slab. Deflections were fairly large.

L-6 with J-10 Expansion Joint. This joint employed a continuous plate dowel 3/16 in. x 1½ in. In the tests of the specimens containing this device, rotation of the dowel plate exerted a splitting effect on the slabs, starting cracks which rapidly produced complete failure. The cracks ran to the top of the middle slab and the bottom of the end slabs, breaking off triangular sections the whole width of the slab. There was also evidence of twisting of the slab and unequal deflection across the slab width.

In two of these slabs, cracks developed and failure occurred at 20,500 and 22,500 lb., respectively. Then, due to wedging of the center slab against the two end ones, the specimens took more load, final failure occurring at 40,200 and 39,000 lb., respectively. It was believed, however, that this was not representative of actual conditions, where such accidental wedging could not maintain itself for any length of time. The initial failures were considered to be the actual failures of the joints.

L-10 with J-7 Expansion Joint. The load transmission units consisted of steel assemblies encasing ¾-in. steel dowel pins. Two features of this joint should be noted. The company stated that they unintentionally furnished the joints 1 in. wide and dowel pins of rather soft screw stock. Both of these factors might be expected to result in low strength in this test. In the test, few cracks appeared until the load approached the maximum. The cracks started from the edge of the copper seal and extended both upward and downward. In some cases, the cracks extended behind the load transmission device, separating it from the remainder of the slab.

L-11 with Wood Joint. The load transmission device used in these tests consisted of malleable castings encasing a ¾-in. steel dowel pin. No joint was supplied with these dowel units, but they were mounted on a ¾-in. pine board for the purpose of casting the concrete slabs. No copper seals were used in these specimens. The boards were left in place during the tests on the first two specimens. It was observed that with this practice additional shearing strength was developed because, under heavy loads, the end sections of the specimen rotated relative to the center section and pressed against the boards. In the first test, for example, although there was general cracking at a load of 38,000 to 39,000 lb. and the specimen yielded and cracked badly at 60,000 lb., the friction developed by the lateral pressure against the boards was sufficient to require a maximum load of 76,500 lb. for complete failure. The action of the other specimen tested under the same procedure was quite similar and the maximum load was 77,000 lb.

The remaining three specimens were tested with the boards removed. There was little cracking of the concrete before the maximum load was reached. When cracks formed they allowed the embedded castings to crush and split the surrounding concrete, sometimes to the extent that the castings were broken entirely free of the concrete. The maximum loads at complete failure were 32,000, 42,600, and 34,000 lb., respectively, or an average of 36,200 lb. The wide difference between these results and those obtained from the first two specimens indicates the magnitude of the error introduced by leaving the boards in place.

A sixth specimen, not included in Table 4, contained devices with pins made from higher strength steel. It was tested with the boards removed, failing similarly to the other specimens tested in that manner, but at a higher load, the maximum being 48,000 lb.

J-9 Expansion Joint with Round Dowels. These joints were equipped with 1-in. round dowels, 19 in. long. Failure evidently was due to crushing and splitting of the concrete under the dowels in the end slabs and above the dowels in the center slab. The latter was very noticeable, since cracks ran from joint to joint on the top of the center slab, parallel to the dowel bars. This is surprising, considering that the stiff loading plate applied a uniform load over most of the middle slab.

J-9 Expansion Joint with Oval Dowels. The oval dowels were submitted as an alternate for the 1-in. round dowel, which was understood to be standard for the J-9 joint. The dowels, evidently made from tubing, were 1 in. x 1¾ in. outside, with walls 0.15 in. thick. They were used with the major diameter horizontal. Three specimens containing oval dowels 17, 19, and 21 in. long, respectively, were tested. Cracks developed as the ultimate load was approached, indicating crushing and splitting of concrete under and above dowels.

L-15 with Premolded Joint. As noted before, half of these joints were tested with the central vertical plate of the joint in tension and half with the plate in compression. The tests showed that the plate would provide about 35 per cent more load transfer when under tension than when under compression. Since traffic loads would cause both types of stress, the minimum strength evidently represents the usable strength of the joint. The joints were opened about ⅛ in. in these tests. If they were opened more, as might happen in service, the load transfer might be further reduced, due to the tendency of the vertical plate to buckle under compression. These joints generally failed by straightening of the anchor lugs and splitting off the edge of the supporting concrete slabs or of the central suspended slab. The average maximum loads with the plate in tension and in compression were 23,450 and 17,075 lb., respectively.

L-16 with Premolded Joint. This device carried a very high load in the load transmission test. The average value of 66,900 lb. on four units, or 16,725 lb. per unit, was the highest found for any dowel unit and is about 2¼ times that developed by the common ¾-in. round dowel bar. However, the device can transfer load in only one direction. It weighs 4.6 lb., as compared to 3.0 lb. for the ¾-in. bar, 24 in. long, which can transfer load in either direction; on the basis of weight the load transfer of the conventional dowel is better than that of the L-16 device.

The specimens failed by cracking and splitting of the supporting slabs, some distance away from the joint. All of the units were slightly bent at the maximum load, and in one case a unit broke in the bend of the flattened portion at the bottom edge of the supporting slab.

Plain Concrete Slabs Initially Cracked. For comparison with the various types of mechanical load transmission devices, tests were made on plain concrete slabs to see what load could be carried by slabs initially cracked and then held in contact and loaded as the other test slabs were. Three plain slabs were made, 23½ in. wide and 7 in. deep. To aid in forming the crack, a groove about ¼ in. deep was formed with an edging tool. The slabs were then cracked with this groove on the tension surface. One of the three broke so irregularly that it could not be tested. The other two were clamped together with the cracks opened only slightly.

In the first test the initial cracks were very nearly vertical. The central slab was held quite firmly by the irregularities of the cracked surfaces, which formed the so-called "aggregate interlock." Failure occurred by diagonal cracks developing in the lower part of the supporting slabs, evidently due to diagonal tension, at a maximum load of 85,550 lb. In the second test, the cracks were inclined about 1 in. to the vertical, in order to make the central slab wedge slightly, like a keystone. Failure was by diagonal tension in the lower part of the supporting slabs and one upper edge of the central slab, at a load of 93,100 lb.

Plain Concrete Slabs with 2-in. Dummy Joints. Three slabs, identical with the plain slabs described above but with a dummy joint at the regular joint position, were tested. The dummy joints, formed by use of a wood strip, were 2 in. deep, 3/8 in. wide at the bottom, and 1/2 in. wide at the top. They were placed in the upper 2 in. of the 7-in. x 23½-in. slab. Before the slabs were tested to determine the shearing resistance of the joint, they were cracked and then clamped together with the cracks open about 1/16 in.

The first specimen was cracked so as to produce some wedging effect when the central slab was loaded. The lower corners of the supporting slabs failed in diagonal tension at a load of 95,700 lb., which was relatively high. The second specimen was so cracked as to produce a non-wedging taper on one side of the central slab, and very large deflections resulted on this side. The maximum load of 46,000 lb. indicates the effect of an unfavorable direction of the initial cracks. The third specimen had initial cracks that were nearly vertical, but there seemed to be some pivoting about a diagonal axis. Failures similar to those in the first specimen occurred at a maximum load of 69,700 lb.

Concrete Slabs with Dummy Joints and Wire Mesh Reinforcement. These three slabs had dummy joints like those described above, but in addition were reinforced with one layer of welded wire mesh having No. 4 wires at 6-in. spacing in the direction of the span and at 12-in. spacing in the lateral direction. This amount of mesh is equivalent to 44 lb. per 100 sq. ft. of pavement. The mesh was placed at the mid-depth of the 7-in. slab, and the dummy joints extended 2 in. below the top surface.

Care was taken, preliminary to the shearing test, to crack these joints entirely through the section. However, when the load producing the crack was removed, the tension in the reinforcement acted to close the crack tightly, leaving only a hairline crack of perhaps 0.005 to 0.01 in. in width. This was evidently responsible for the high loads of 110,700, 128,300 and 106,400 lb. developed by the three slabs.

Failure of these slabs was in general similar to that of the plain concrete slabs; i.e., due to diagonal tension and shear cracks in the lower part of the supporting slabs. The aggregate interlock at the original cracks seemed to be very effective.

(3) Comparison of Results. Load transmission devices are generally considered to have two functions: (1) to keep slab edges in line and to preserve a smooth riding surface; and (2) to transfer a portion of a wheel load across a joint to the adjacent slab and thus reduce the stress in the slab.[1] Since the vertical deflections of a road slab on ordinary subgrade are very small, it is evident that a load transmission device, to be effective, must be rigid and allow a minimum of play between its parts so that it will permit only a few thousandths of an inch differential deflection between the two slabs which it connects.

Another feature to be considered in connection with load transmission device deflections and displacement is that whenever a wheel passes over a joint a reversal of shear occurs. Thus, as a wheel approaches the joint, the

[1] See discussion on "Effect of Dowel-Bar Misalignment Across Concrete Pavement Joints," by L. W. Teller, Proc. A.S.C.E., pp. 1632-4, October, 1937.

device transfers a portion of the load by a *downward bearing* on the adjacent slab. As the wheel passes to the adjacent slab, the shear in the device is reversed and the device *lifts upward* on the loaded slab. This reversal of stress and deflection means that a large amount of play in the device will do one of two things: (1) it may produce a hammering or impact effect tending to loosen the device, commonly known as funneling; or (2) if the movement is too large the device will be ineffective in transferring shear. Large differential deflections at a joint are perhaps responsible for much of the tearing of the copper seals in service.

In analyzing the data obtained from the load transmission tests, average load deflection and permanent set curves were plotted for each test specimen, using the average of the deflections obtained at four points on the specimen, two on each joint. From the individual curves for each type of load transmission device or joint, a single composite curve was drawn. Composite curves for each type of load transmission device or joint are shown in Figs. 53 and 54. These figures show at a glance the relative stiffness of the various devices as tested. It should be noted, however, that since the joints were not all of equal width direct comparisons of these curves are misleading.

The L-15 load transmission system was used in conjunction with a premolded filler ⅜ in. thick. J-8 and J-9 joints employed premolded fillers ¾ in. thick. The premolded filler in the J-10 joint was ½ in. thick. J-1, J-2, J-3, J-4, J-5, J-6, and J-7 were air-chamber joints. J-1, J-4, and J-5 were ¾ in. wide; J-6 was ½ in. wide; and J-2, J-3, and J-7 were 1 in. wide. Also, J-2, J-3, and J-7 employed filler blocks or spacers of preformed fiber or bituminous joint material instead of metal spacers to prevent collapse of the side walls of the joints, and in this respect did not meet the 1937 specifications of the Illinois Division of Highways. Thus, it appears that only the J-1, J-4, and J-5 joints were designed to meet the Illinois specifications as to the following requirements: (1) air-chamber type; (2) full ¾-in. net expansion space.

It should be noted that the J-6 joint was designed for installation every 30 ft., instead of 90 ft. as specified.

In choosing a criterion as to the amount of deflection that might be considered excessive, it is first necessary to know the probable load to be carried by a single dowel or unit. A study of the shears transferred across a multiple-dowel joint, with the loaded slab suspended between supporting slabs as in these tests, indicates that with the usual 13½-in. dowel spacing about 25 per cent of a wheel load may be carried on a single dowel. With wider spacings the proportions may go to 35 or 40 per cent. If all slabs bear equally on a soil subgrade, it is obvious that the loads transmitted across the joint will not be more than one-half as great as in the case of the suspended slab. Hence, in a pavement the load carried by one dowel might be expected to be approximately 13 to 20 per cent of the wheel load. For the standard H-20 loading, the load per rear wheel is 16,000 lb. Using the average of the above range in percentages, 16 per cent of 16,000 lb. is 2,560 lb., and if this is combined with an overload factor of 2, the critical load per dowel is 5,120 lb., or in round numbers, 5,000 lb.

The deflection characteristics of the various devices tested are given in Table 5 for loads of 2,500 to 5,000 lb. per device. Since the test slabs contained four dowels (or their equivalent, presumably), the total loads considered in Table 5 are 10,000 lb. and 20,000 lb. It is seen from the table that at the load of 10,000 lb., the deflection of the ordinary ¾-in. bar was 0.0069 in.; of the L-1, 0.0077 in.; of the L-13 round and oval dowels, 0.0048 and 0.0038 in.; of the L-4 device (½-in. joint), 0.003 in. The deflections of the L-11 unit were definitely higher than those of the standard dowel at 10,000 lb., being 0.0085 to 0.0105 in. The deflections of the L-2, L-5, and L-6 devices were 2.8 to 3.4 times that of the standard dowel at this load. Compared to these dowel units, the behavior of the plain slab, dummy joint, and wire mesh specimens

TABLE 5
LOAD DEFLECTION DATA FROM LOAD TRANSMISSION TESTS[1]
(University of Illinois Tests)

Kind of Load Transmission Device	Width of Joint	Maximum Load on Slab	At 10,000-lb. Load		At 20,000-lb. Load	
			Deflection	Perm. set	Deflection	Perm. set
	in.	lb.	in.	in.	in.	in.
L-1	¾	48,820	0.0072	0.0016	0.0165	0.0041
	¾	48,700	0.0082	0.0020	0.0189	0.0048
		48,760	0.0077	0.0018	0.0177	0.0045
¾-in. dowel, 24 in. lg.	¾	28,090	0.0068	0.0012	0.0230	0.0062
	¾	30,690	0.0070	0.0010	0.0213	0.0066
		29,390	0.0069	0.0011	0.0222	0.0064
L-4	½	50,380	0.0030	0.0008	0.0089	0.0024
1 in. dowel, 17 in. lg.	¾	51,720	0.0048	0.0014	0.0132	0.0035
Oval pipe	¾	50,960	0.0038	0.0009	0.0105	0.0022
L-10	1	43,180	0.0105	0.0025	0.0257	0.0080
L-11	¾	36,200	0.0085	0.0020	0.0187	0.0055
L-5	¾	34,500	0.0195	0.0030	0.0455	0.0105
L-6 (3/16-in. x 1½-in. plate)	½	21,900	0.0260	0.0110	0.0800[2]	0.0550[2]
L-2	1	16,360	0.0236	0.0044	......	......
L-15 (tension)	⅜	23,450	0.0178	0.0053	Near failure	Near failure
L-15 (comp.)	⅜	17,075	0.0035	0.0007	......	......
L-16	¾	66,900	0.0022	0.0006	0.0064	0.0014
Plain slab[3]	..	89,320	0.0030	0.0019	0.0077	0.0051
Plain slab, 2-in. dummy joint[3]	..	70,670	0.0040	0.0029	0.0112	0.0085
2-in. dummy joint with 44-lb. wire mesh[3]	..	115,130	0.0005	0.0000	0.0014	0.0005

[1] In general five tests were made on each type.
[2] Estimated.
[3] The plain slab, dummy joint and wire mesh specimens, all 7-in. slabs, were cracked before the load transmission test was started.

was noteworthy. All were very stiff and the specimen containing wire mesh, in particular, was six times as stiff as any of the slabs employing dowels.

The deflections under the 20,000-lb. load were still more significant as regards the load-carrying ability of the dowel units. Here again the L-1, L-4, and L-13 were definitely stiffer than the standard ¾-in. dowel, the L-11 device about equal, and the L-5 and L-6 devices much more flexible, while the L-2 device failed completely.

(4) Effectiveness of Load Transmission Units. The joint deflections given in Table 5, together with information on the stiffness of road slab and subgrade, may be used to indicate the relative effectiveness of different types of load transmission units. The simplest comparison to be made is that of the action of the single unit across a joint with a concentrated load on one side of the joint and directly above the device. It is probably safe to assume that the deflection of subgrade under one edge of the slab is unaffected by the deflection under the other edge a short distance away.

If the device had zero stiffness, no shear would be transferred across the joint. If the device were infinitely stiff, 50 per cent of the load would be transferred by shear to the unloaded slab, and we might consider the device as 100 per cent effective. Thus, the effectiveness, e, could be defined as the ratio of twice the shear, V, to the concentrated load, P, or

$$e = \frac{2V}{P}. \tag{1}$$

If the deflection of the device across the joint due to unit shear is Δ, and the deflection of the subgrade due to a unit load on the edge of the slab is z, then, since the difference of the subgrade deflections on the two sides of the joint must equal the dowel deflection,

$$(P - V)z - Vz = V\Delta. \tag{2}$$

Combining Equations (1) and (2) gives

$$e = \frac{1}{1 + \dfrac{\Delta}{2z}}. \tag{3}$$

This equation has also been given by Friberg[2] and Bradbury[3] in discussions of dowel design.

From Equation (3), it is evident that the effectiveness of a dowel in transferring shear depends not only upon its own flexibility as measured by Δ, but also upon the stiffness of the subgrade and the thickness and stiffness of

[2] Friberg, B. F., "Design of Dowels in Transverse Joints of Concrete Pavements," Proc. A.S.C.E., Vol. 64, No. 9, pp. 1809-28, November, 1938.
[3] Bradbury, R. D., Discussion, "Load and Deflection Characteristics of Dowels in Transverse Joints of Concrete Pavements," Proc. Highway Research Board, Vol. 18, Part I, p. 156, 1938.

TABLE 6

COMPUTED THEORETICAL EFFECTIVENESS OF LOAD TRANSMISSION DEVICES FOR THREE VALUES OF SUBGRADE SUPPORT

(University of Illinois Tests)

(Assumed: 7-in. slab, Modulus of Elasticity = 3,000,000 lb. per sq. in., Poisson's ratio = 0.15.)

Kind of Load Transmission Device	Width of Joint in.	Load per ft. of Joint, lb.					
		2,500			5,000		
		Compressible soil k = 50	Medium soil k = 100	Stiff soil k = 250	Compressible soil k = 50	Medium soil k = 100	Stiff soil k = 250
L-1	3/4	0.82	0.77	0.71	0.80	0.74	0.68
L-1	3/4	0.80	0.75	0.68	0.78	0.72	0.65
3/4-in. dowel	3/4	0.83	0.78	0.72	0.74	0.68	0.60
3/4-in. dowel	3/4	0.83	0.77	0.71	0.76	0.69	0.62
L-4	1/2	0.92	0.89	0.85	0.88	0.84	0.80
1-in. dowel	3/4	0.87	0.83	0.78	0.83	0.79	0.73
Oval pipe	3/4	0.89	0.86	0.82	0.86	0.82	0.77
L-10	1	0.76	0.70	0.62	0.72	0.65	0.57
L-11	3/4	0.80	0.74	0.67	0.78	0.72	0.65
L-5	3/4	0.63	0.55	0.47	0.59	0.51	0.43
L-6 (3/16-in. x 1 1/2-in. plate)	1/2	0.56	0.49	0.40	0.45	0.38	0.30
L-2	1	0.59	0.51	0.42	....	....	....
L-15 (tension)	3/8	0.65	0.57	0.49	....	....	....
L-15 (comp.)	3/8	0.91	0.87	0.83	....	....	....
L-16 (12-in. effective spacing)	3/4	0.94	0.92	0.89	0.84	0.80	0.73
L-16 (24-in. effective spacing)	3/4	0.84	0.80	0.73	0.60	0.52	0.44
Plain slab, with wire mesh	..	0.99	0.98	0.97	0.98	0.97	0.96

the road slab, as reflected in the deflection, z. This quantity may be computed for given conditions, by means of Westergaard's[4] equation

$$z = \frac{0.433\, P}{kl^2} \tag{4}$$

in which z is the deflection of a free edge of a slab, directly under the concentrated load P, k is the modulus of subgrade reaction, and l is the radius of relative stiffness for the slab. For a 7-in. pavement, with a modulus of elasticity for the concrete of 3,000,000 lb. per sq. in., k and l may have the following values.

Compressible soil.............. $k = 50$ $l = 36$
Medium soil.................. $k = 100$ $l = 30$
Stiff soil...................... $k = 250$ $l = 22$
Very stiff soil................. $k = 500$ $l = 20$.

Table 6 gives values of the relative effectiveness of load transmission devices in a 7-in. slab, computed for subgrade moduli of 50, 100, and 200 lb. per cu. in. and loads of 2,500 and 5,000 lb. per foot of joint, corresponding roughly to one and two times the normal service load on a slab under H-20 loading. These were computed by substituting in Equation (3) values of de-

[4] Westergaard, H. M., "Computation of Stresses in Concrete Roads," Proc. Highway Research Board, pp. 90-112, 1926.

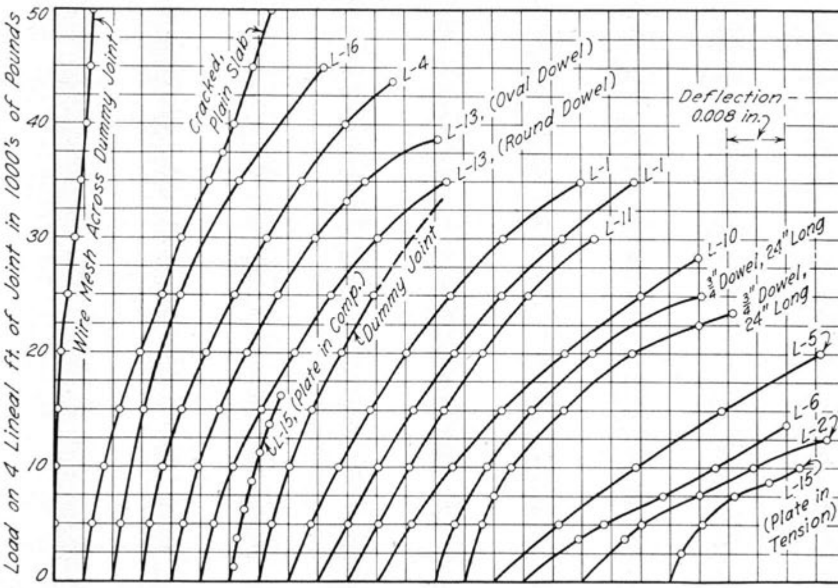


FIG. 53. AVERAGE LOAD DEFLECTION CURVES, LOAD TRANSFER TESTS

flection, z, obtained from Equation (4), using proper values of k and l. The results show that even the best of the load transmission devices are considerably less than 100 per cent effective in transferring load across a joint supported on subgrade. The effectiveness is evidently greater for the more compressible soils, and this is fortunate, because even though a device is less effective on a very stiff soil, such a subgrade results in small deflection, and therefore small stresses, in the pavement slab under a wheel load. Attention may therefore be concentrated on the effectiveness of a device in slabs on soft and medium soils.

The computed relative efficiencies of the joints agree reasonably well with the comparison on the basis of deflections discussed above. On the basis of relative efficiency, the various mechanical devices are rated in the following order, beginning with the most efficient: L-16, L-4, J-9 with oval dowel, J-9 with round dowel, L-1, conventional ¾-in. dowel, L-11, L-10, L-15, L-5, L-2, and L-6. The relative stiffness and strength of the various mechanical devices are also shown by Fig. 53, in which the L-16, L-4, J-9 with oval dowel, J-9 with round dowel, L-1, and L-11 devices fall into a fairly definite group, while the remaining seven devices show consistently higher deflections, especially at the higher loads. Figure 54 shows average load set curves.

It has been noted that the L-16 device can transfer load in only one direction across a joint. The high computed effectiveness of this joint is therefore based on the use of two units (one facing each way) per linear foot of joint. If the dowels are alternated in position and used at a 12-in. spacing per unit,

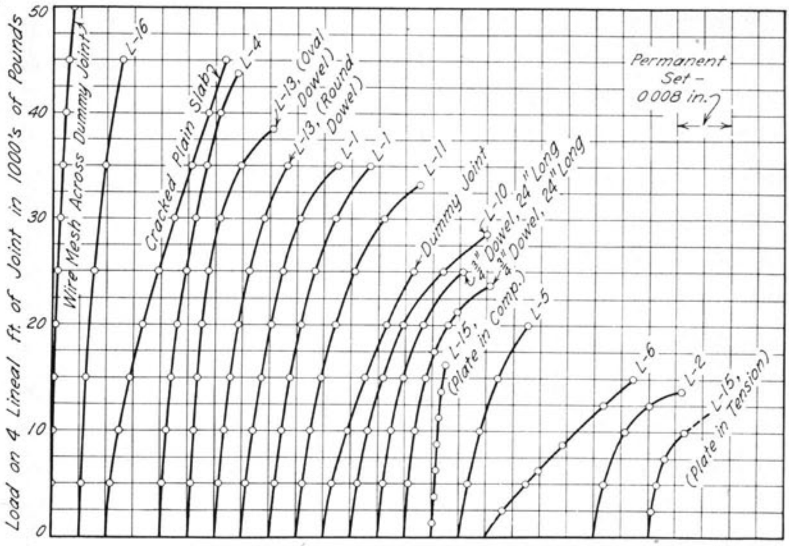
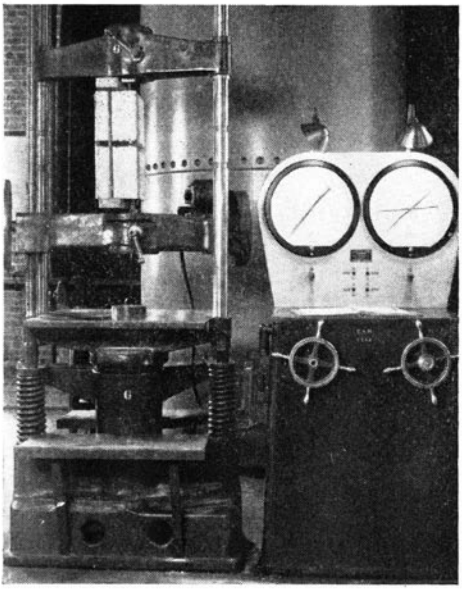


FIG. 54. AVERAGE LOAD SET CURVES, LOAD TRANSFER TESTS

the effective spacing becomes 24 in. and the resulting effectiveness is considerably decreased, as shown in Table 6.

The numerical values of effectiveness of load transfer in Table 6 should not be given too much weight. They show clearly the need for stiffness as well as strength in a load transmission device. They do *not* show how this effectiveness may be modified in service by loosening of the unit, funneling of concrete around the device, softening of the subgrade due to leakage or infiltration of water into the joint, or curling of the pavement. Many engineers have pointed out that with curling of the slab, due to temperature and moisture differentials, a stiff load transmission device may crack the slab by its inability to bend with the slab. The ideal device for this purpose should provide hinge action at the joint. The laboratory tests did not produce much bending in the dowel joints; further tests with large amounts of bending are needed for a complete answer to the curling problem. To the extent that a load transmission device reduces the deflection of the road slab at a joint from the amount that would exist if there were no load transfer, it may also be considered effective in reducing the stresses in the slab. However, since the load in service is not usually applied directly over a load transmission device, and since the load must be carried by a group of units, the effectiveness as a load transmission device does not indicate directly the effectiveness of the unit in reducing slab stresses. This is particularly true if a comparison is made between two types of devices, used at different spacings. This subject has been discussed by Friberg,[5] Bradbury[6] and others, and need not be developed further here. However, it is worth

[5] Friberg, B. F., "Design of Dowels in Transverse Joints of Concrete Pavements," Proc. A.S.C.E., Vol. 64, No. 9, pp. 1809-28, November, 1938.
[6] Bradbury, R. D., Discussion, "Load and Deflection Characteristics of Dowels in Transverse Joints of Concrete Pavements," Proc. Highway Research Board, Vol. 18, Part I, p. 156, 1938.



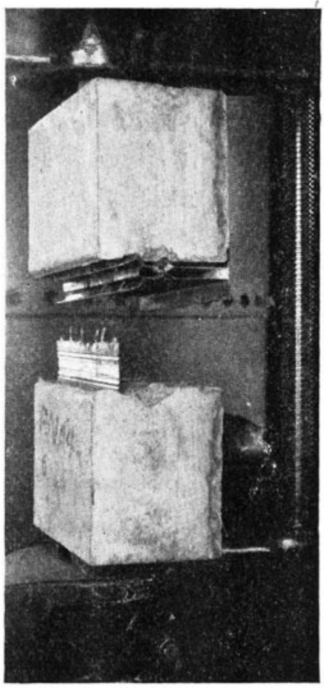


FIG. 55. ARRANGEMENT OF PULL-OUT TESTS OF COPPER SEALS. (a) ARRANGEMENT OF TEST PIECE IN TESTING MACHINE (CLAMP SHOWN IS FOR HANDLING TEST PIECE); (b) TEST PIECE AFTER FAILURE OF SEAL (UNIVERSITY OF ILLINOIS TESTS)

emphasizing that, as a means of comparing joints of different strengths and stiffnesses, the effectiveness factors given in Table 6 seem very useful.

(e) Tests of Anchorage of Copper Seals

(1) TEST ARRANGEMENT AND PROCEDURE. The copper seals used on the various joints were anchored into the adjacent concrete slabs by edges, beads, or wings projecting from 3/16 in. to 1¼ in. from the face of the joint. To secure information on the effectiveness of such anchorage, a series of tests was made in which a 1-ft. length of the joint (generally without dowels) was embedded in a small concrete slab. After a two-week curing period the two halves of the slab were pulled apart, causing the copper seal first to straighten out, then to tear in two or pull out of the concrete. Five such "pull-out" tests were made with each type of joint.

Figure 55 shows the arrangement of a pull-out test in a testing machine. The slab, 12 in. wide and 7 in. thick, was set vertically in the machine. A pull was applied to the lower block, in line with the plane of the copper seal anchors, and the joint was pulled apart until the copper pulled out of the concrete or broke. The loads required to produce significant action of the joint were recorded.

TABLE 7
RESULTS OF PULL-OUT TESTS OF COPPER SEALS
(University of Illinois Tests)

Joint	Specimen	Load at Initial Slip lb.	Maximum Load lb.	Average Maximum Load lb.	Remarks
J-1	1	2,300	2,300		Bottom failed, then seal tore out along slotted holes.
	2	600	2,140		Bottom failed, then seal tore out along slotted holes.
	3	1,840	1,920		Bottom failed, then seal tore out along slotted holes.
	4	2,120	2,180		Bottom failed, then seal tore out along slotted holes.
	5	2,100	2,200	2,148	Bottom failed, then seal tore out along slotted holes.
J-4	1	1,950	2,200		Concrete cracked above seal.
	2	1,600	2,200		Concrete cracked above seal.
	3	1,900	2,200		Concrete cracked above seal.
	4	1,750	2,450		Broke out block of concrete, one end.
	5	1,850	2,380	2,286[2]	Crack formed near edge of seal.
J-5	1	2,400	2,600		Seal torn and concrete broke at corner.
	2	2,000	2,600		Seal slipped at one corner.
	3	2,250	3,150		Concrete split off above seal.
	4	2,100	3,540		Concrete split off above seal.
	5	2,540	2,820	2,994[2]	Seal pulled out and split concrete.
J-6	1	1,300	1,700		Slipped and broke 2 copper lugs.
	2	1,950	2,100		Broke off 3 copper lugs.
	3	2,600	2,600		Lugs broke and seal slipped out.
	4	1,000	2,100		Lugs broke or pulled out.
	5	2,400	2,400	2,180	Seal slipped out of concrete.
J-7	1	1,360	1,360		Failed at steel strap across seal.
	2[1]	1,560	1,560		Failed first at one corner.
	3[1]	1,590	1,590		Same as No. 2.
	4	1,240	1,240		Same as No. 1.
	5[1]	1,560	1,560	1,462	Cracked loose seal at one corner, then the other.
J-2	1	980	1,300		Cross tie broke at 980 lb.
	2	1,150	1,400		
	3	1,650	1,650		Concrete cracked, copper seal tore.
	4	1,750	2,200		Similar to No. 3.
	5	2,070	2,070	1,724	Cracked concrete one corner, slipped at other.
J-3	1	1,680	1,680		Failed at corner by cracking of concrete.
	2[1]	1,720	1,720		Seal pulled out, edge of slab split.
	3[1]	1,130	1,130		Failure at corner, concrete cracked.
	4	1,470	1,470		Failure at corner, concrete cracked.
	5[1]	1,180	1,180	1,436	Sudden failure, concrete broke out.
J-10	1	900[3]	920		Concrete split out above seal.
	2	950[3]	950		Joint spread, concrete split at upper seal.
	3	840[3]	840		Joint spread, seal pulled out.
	4	2,300[3]	700		Steel plate holding seal yielded.
	5	1,060[3]	800	842	Steel plate loosened, releasing seal.
J-8	1	2,000	2,000		Concrete cracked. Wire anchor opened up.
	2	1,920	1,920		Concrete cracked. Wire loop tore thin copper seal.
	3	1,500	1,500		Concrete cracked. Wire loop opened.
	4	1,800	2,300		Concrete cracked above seal.
	5	1,400	1,660	1,876[2]	Concrete cracked. One wire loop opened, other tore seal.
J-9	1	1,800	2,020		Seal pulled out of concrete.
	2	1,550	3,180		Steel strap broke, seal pulled out.
	3	1,150	2,750		Steel strap broke, concrete cracked vertically.
	4	570	570		Concrete split, seal pulled out.
	5	1,280	2,760	2,256[2]	Concrete split, seal pulled out.

Note: Slabs 12 in. wide, 7 in. deep. No dowels used except the plate in the J-10 joint, 4,000-lb. concrete. Joints pulled apart until copper seal straightened and pulled loose from concrete.

[1] The copper seal was tied at intervals with a steel strap, which produced failure by concentrated bending of copper and cracking of concrete. On slabs J-7 (2, 3, and 5) and J-3 (2, 3, and 5), this strap was cut before the test.

[2] Copper seal anchors backed up and reinforced with steel sheet.

[3] Load at opening of J-10 unit.

(2) Data of Pull-Out Tests. The principal test results are given in Table 7. A description of the manner in which each seal failed is given below:

J-1. This joint generally failed by first pulling out the bottom of the air chamber, and then tearing the copper seal along the row of slotted holes, leaving the outer bead embedded in the concrete.

J-4 and J-5. In these joints the outstanding flange of the copper seal was crimped to a similar flange on the sheet steel body of the joint. Failure was due to cracking and splitting of the concrete above the seal, allowing the copper and steel flange to pull out.

J-6. This seal had projecting lugs ½ in. wide, spaced at 2-in. intervals along the length of the copper seal. In the tests the joints generally failed by breaking off these copper lugs and consequent slipping of the remaining seal from the concrete.

J-3 and J-7. The seals on these joints were almost identical in design and in behavior in the tests. The M-type copper seal was tied together by means of a light steel strap at intervals of about 10 in.; when these straps were left in place they caused an initial local splitting of the concrete. In part of the tests the straps were cut, allowing the copper joint to open and straighten out freely. The final strength of the joint was not greatly affected by the steel straps.

J-2. This joint failed by cracking of concrete and tearing or pulling out the copper seal.

J-10. This joint had an unsymmetrical seal, one side projecting ½ in. with a rolled bead, and the other hooked over a light steel plate and projecting with a straight edge 3⁄16 in. into the concrete. This narrow edge failed in most cases, since the steel plate yielded enough to allow the copper to pull out of the concrete, at a relatively low load.

J-8. This copper seal was reinforced or backed up with a sheet of steel which added to its strength and stiffness. The ½-in. projection into the concrete consisted of two thicknesses of copper and one of steel. To increase the anchorage, No. 10 wire dowel holders were hooked through the copper seal at intervals. At failure, there was cracking of the concrete around the seal, and the wire loops either opened up or tore out of the seal.

J-9. This seal, with straight edges projecting about 9⁄16 in. into the concrete, was greatly strengthened by steel straps used to support the seal in place. With one exception, this seal carried a rather high load, for which the steel straps were doubtless partly responsible.

When these tests were planned, there was some doubt that concrete could be placed against and around the copper seals in a way that would insure good anchorage of the seals. Unless the seals in a road slab remain thoroughly anchored and unbroken, they can have no permanent value. Therefore, while no joints in service would open sufficiently to pull out the seals, as was done in this test, conditions may occur in a pavement where the asphalt filler or dirt is pounded tightly against the vertical wall of the seal, which exerts a heavy pull on the anchorage of the copper.

The tests may also indicate other weaknesses of the sealed joint, such as honeycombed concrete beneath the wings of the seal, or a weakness in the thin ledge of concrete just above the seals. Another feature of certain seals is suggested by these tests; viz., when the seal anchors consist of combined layers of copper and steel, the permanent resistance of the anchorage to corrosion is doubtful.

These tests, while not so important as other tests of the joints, seem sig-

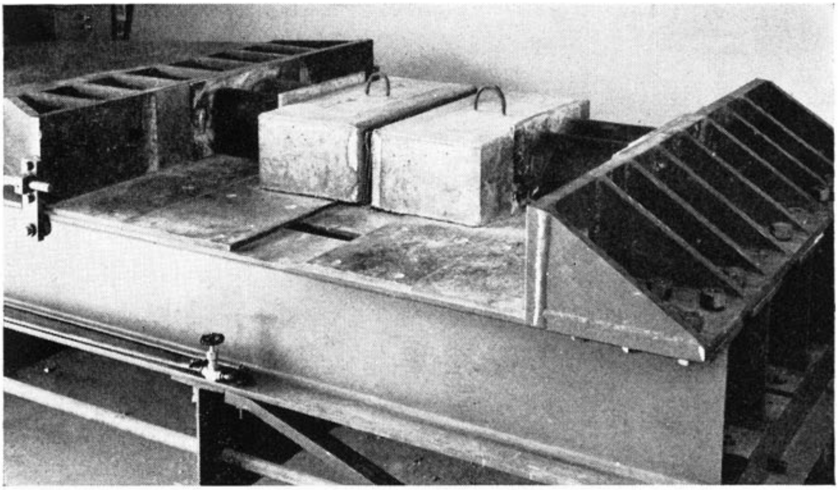


FIG. 56. HYDRAULIC TESTING MACHINE FOR OPENING-CLOSING TESTS (STATE HIGHWAY LABORATORY, SPRINGFIELD)

nificant in showing the relative security of anchorage of the several types of copper seals.

(f) Repeated Opening-Closing or Fatigue Tests of Joints

(1) TEST ARRANGEMENT AND PROCEDURE. The object of this series of tests was to determine the ability of the joint, and particularly of the copper seal, to withstand repeated opening and closing, such as might be produced by expansion and contraction of a pavement due to changes in temperature. The 1937 specifications of the Illinois Division of Highways required that the joint should have a net expansion space between the side walls of not less than ¾ in. With reference to the copper seal, it was specified that under laboratory tests it must withstand, without injury in any way, not less than 40 cycles of alternate complete opening and closing of the joint. It might be assumed that while there are daily variations in pavement length, a complete opening and closing of the joint would occur only once a year, so that 40 cycles might be considered to be equivalent to 40 seasons' variation. However, it is more likely that the daily variations in summer, when the joint is most tightly closed, and the accompanying vertical deflections across the joint, do more damage to the copper seal than has generally been assumed.

The tests were made under the supervision of Professor Dolan on a special machine designed for the purpose, which was available at the State Highway Laboratory at Springfield. By means of this machine, a 24-in. section of joint, embedded in a slab of concrete, was subjected to a very gradual opening and closing, with about one cycle per minute and with any desired amount of movement. The test slabs, 24 in. square and 7 in. thick, were tested when seven days old. The machine, shown in Fig. 56, was operated hydraulically with an automatic mechanism to reverse the direction of motion. After a test was begun, it was stopped at regular intervals to permit a thorough examination of the copper seal in order to determine the extent of cracking produced. The machine was provided with gages which indicated, approximately, the load required to close or open the joint.

When possible, the joints were subjected to a complete ¾-in. closing, as required by the 1937 specifications, but since some joints would not close that much, they were simply closed as tightly as their construction permitted. Included in the group which could not be closed ¾ in. were the J-6 joint with ½-in. air chamber and the J-8, J-9, and J-10 joints which contained fiber fillers. In the last three joints, the copper seal received much less severe treatment than the others, since the closure was held to only 5⁄16 to ½ in., and, in addition, the extrusion of the filler formed a cushion inside the seal which prevented sharp folds in the top of the seal.

All tests were run to at least 100 cycles of opening and closing, unless general failure of the top seal had occurred at a lower number of cycles. For a few joints, the number of cycles reached 200 to 300.

For the joints containing preformed asphalt spacer blocks (J-2, J-3, and J-7), a fairly high load was required for the initial closing of the joint, but the asphalt extruded readily, the joint closed almost completely, and the loads at subsequent cycles were low.

An excessive amount of solder in the lap seams, at the junction of top and end seals of some joints, usually caused an early failure of the seam and in some cases started cracks in the copper near the seam.

No dowels were used in these tests, except for the continuous plate dowel of the J-10 joint. On joints furnished with asphalt caps, the cap was removed from the copper seal before the test.

(2) Data of Opening-Closing Tests. Detailed records of each test were kept, with notes and sketches showing how cracks in the seals started and progressed with repeated loading. The principal items from these records, for all of the joints tested, are presented in Table 8. As a further summary and generalization of the results, Table 9 gives a brief classification of observed defects of the various joints. It will be noted from a reference to the tables that in all but three types of joints the soldered junction between top and end seal broke the first time the joint was closed. All but four had cracks more than 1 in. long after 40 cycles of movement. Three types required very large loads to produce even partial closure of the joint. Only one joint was fairly free from the defects listed, and it certainly did not withstand 40 cycles of movement *without injury in any way*.

(3) Detailed Test Behavior of the Various Joints. The following detailed notes describe the test behavior of the ten joints listed in Table 8:

J-1. This joint was one of the few that permitted the full ¾-in. closure without folding the seal tightly. The seal was effective in preventing any mortar from entering the air chamber. During the first 50 cycles the cracks in the seal, which were very small, were due to breaking of the brazed lap at the junction of end and top seal. After a large number of cycles, small cracks started in the end

TABLE 8

RESULTS OF OPENING-CLOSING TESTS ON COPPER SEALS
(University of Illinois Tests)

Kind of Joint	Speci-men No.	Total Closure	Maximum Load	Number of Cracks of Cycle No.				Maximum Length of Crack in Inches at Cycle No.				Number of Cycles Applied	Remarks
		in.	lb.	1	20	40	100	1	20	40	100		
J-1	1	¾	14,000	..	4	7	8	..	¼	⅝	1½		Cracked first through solder.
J-1	2	¾	1,000	..	3	4	5	..	⅛	⅝	1¾	200–2½-in. crack	
J-1	3	¾	20,000	..	1	8	11	..	⅛	¼	1½	360–4½-in. crack	Asphalt cap left on for 10 cycles.
J-10	1	5/16	190,000	..	..	..	5/285	..	..	..	..	285–6-in. crack	Concrete broke at end of joint above end seal.
J-10	2	11/32	180,000	..	..	..	2/160	..	..	..	..	160–⅜-in. crack	
J-10	3	5/16	190,000	..	..	..	..	..	..	..	..	240–No cracks	
J-8	1	7/16	180,000	2	4	6	8	1/16	½	¾	1¾		Soldered joints broke at cycle 1.
J-8	2	13/32	175,000	2	5	7	7	½	½	¾	1	150–Failed[1]	
J-8	3	13/32	120,000	2	7	9	12/108	¾	¾	¾	1½	150–Failed	
J-9	1	15/32	95,000	2	3	4	..	¾	13	13	..	87–Failed	Top corner opened on cycle 1. Failed at sharp corner entrance to concrete.
J-9	2	15/32	125,000	2	3	4	..	¾	11	15	..	47–Failed	
J-9	3	½	172,000	2	2	4	5	¾	¾	8	18½	102–Failed	
J-6	1	7/16	75,000	..	..	..	4	..	..	..	⅞	190–7-in. crack	Mortar in joints NA; seal wrinkled.
J-6	2	7/16	100,000	..	1	1	6	..	¼	¾	1	245–3-in. crack	
J-6	3	15/32	110,000	..	..	..	2	..	..	..	⅜	257–12-in. crack	
J-7	1	¾	55,000	2	6	9	12	1/16	1	1¼	6		Mortar in air chamber.
J-7	2	23/32	60,000	2	3	5	11	1/16	½	½	1½	164–Failed[1]	
J-7	3	¾	50,000	3	9	11	14	⅜	1	3	13	100–Failed	
J-7	4	25/32	85,000	2	6	10	13	1/16	⅜	1	3	160–6½-in. crack	
J-4	1	11/16	70,000	2	3	5	7	½	1½	2	3¾	135–4¼-in. crack	Many failures in end seal.
J-4	2	23/32	90,000	2	5	5	7	¾	1¼	3	4½	170–8-in. +4-in. top	
J-4	3	23/32	64,000	1	5	7	9	¾	2	4½	6½	115–7-in. crack	
J-2	1	27/32	50,000	3	8	10	..	3/16	3	6	..	87–Top failed[1]	Middle trough of seal failed starting at diagonal corners.
J-2	2	13/16	90,000	5	12	14	..	¼	1	5	..	60–7 +-in. crack	
J-2	3	27/32	87,000	6	7	8	11	¾	¾	4	17	100–only 4-in. left	
J-2	4	¾	13,000	4	12	12	15	¼	1½	3	11	141–Failed[1]	
J-5	1	¾	1,000	..	6	9	..	..	18	20	..	60–Failed	Failed at sharp corner entrance to concrete.
J-5	2	¾	1,000	1	8	16	..	1/16	¾	9	..	52–Failed	
J-5	3	¾	2,000	1	5	9	..	⅛	3	18	..	61–Failed	
J-3	1	¾	22,000	2	4	7	13	1/16	⅛	½	8	145–8½-in. crack	End seals opened in fold.
J-3	2	23/32	45,000	..	4	4	14	..	1/16	⅛	4¾	147–15-in. crack. Failed[1]	
J-3	3	¾	40,000	2	4	5	11	1/16	¾	1½	6	150–Failed[1]	

[1] Joints marked "Failed" had a practically continuous crack through the copper seal.

TABLE 9
OBSERVED DEFECTS OF JOINTS AND SEALS IN OPENING-CLOSING TESTS
(University of Illinois Tests)

Defect	Joint									
	J-1	J-10	J-3	J-8	J-9	J-6	J-7	J-4	J-2	J-5
Mortar leaks in under top seal	..	..	x	..	..	x	x	..	..	..
Mortar leaks in at bottom corner, end seal	..	..	x	..	..	x	x	x	x	x
Seal folds tightly as joint is closed	..	..	x	..	x	..	x	x	x	x
Joint not closed ¾-in.	..	x	..	x	x	x	..	x	..	..
Actual closure =	..	5/16-in.	..	7/16-in.	½-in.	7/16-in.	..	11/16-in.	..	..
Soldered seams break at first cycle	..	..	x	x	Tape opens	..	x	x	x	x
Large load required to close joint	..	x	..	x	x	..	..	..	..	..
Sharp bends in copper cause large cracks to form within 40 cycles	..	..	..	..	x	..	..	..	..	x
End seal causes concrete to spall	..	x	..	..	..	..	..	..	..	..
Cracks in copper more than 1-in. long after 40 cycles	..	..	x	..	x	..	x	x	x	x
Seals fail completely with 160 cycles or less	..	..	x	x	x	..	x	x	x	x

seals at their junction with the concrete. More than 200 cycles were required to develop a 2½-in. crack in the copper of the top seal. In general, the cracks in this seal did not develop rapidly; the longest crack noted was 4½ in. after 360 cycles.

J-10. This joint permitted a closure of only 5/16 in., hence the copper seal was not folded tightly enough to produce severe flexing of the copper at localized points, as was the case in most of the other joints. For this reason, the joint may be rated as the most resistant to cracking of the group tested. In one test, no cracks were found after 240 cycles of test. However, the J-10 does not provide for ¾ in. of closure. The end seals (sponge rubber and galvanized iron plates) excluded mortar efficiently, but rotated as the joint opened and closed so as to spall off a small amount of concrete above the seal.

J-8. This joint had soldered corner joints which broke open at the first closing. The first cracks in the copper itself started in the sharp folds close to the soldered corners. Since these joints could only be closed 7/16 in., the top seal did not fold very tightly, but with repeated flexing, cracks started and developed along the fold which formed along the middle of the top and end seals. None of these cracks was more than ¾ in. long at 40 cycles or 1¾ in. long at 100 cycles, but thereafter, probably due to cracking of the steel sheet which reinforced the copper, cracking proceeded rapidly to complete failure at about 150 cycles.

J-9. The end seals furnished with this joint, evidently hand-made, were much thinner than the top seals (0.015 in. as compared to 0.026 in.). Unlike those of the other joints, these end seals were set flush with the sides of the slab. Instead of a soldered joint between top and end seals, the junction was covered with electricians' friction tape. In the tests, this tape allowed the corner seal to buckle and open up the first time the joint was closed. Because of the fiber filler, this seal could only be closed ½ in. The top seal failed by a rapidly spreading crack along the sharp bend in the seal at its junction with the concrete. At 40 cycles of testing this seal had cracks from 8 to 15 in. long, and failed completely at about 100 cycles.

J-6. This joint was intended for ½-in. closure but, due to metal separators and other obstructions, could be closed only 7/16 in.; hence the middle portion of the copper seal did not fold tightly. However, in the curved sections joining the top and end seals, many small diagonal wrinkles formed and started cracks as the test progressed. The large cracks which developed in the seal after a large number of cycles of test were generally at the sharp bend in the copper where it entered the concrete slab.

It was found that, during pouring of the fresh concrete, the seal did not prevent mortar from leaking into the air chamber, collecting at the bottom of the joint, and interfering with complete closing.

J-7. The copper seal used on this joint was similar to that used on the J-3 and J-4 joints. In all three, the soldered joint at the junction of top and end seals opened up at the first cycle of testing. The asphalt spacer blocks did not prevent full ¾-in. closing of the joint; neither did the mortar which had leaked into the air chamber. The cracks in the seal were generally small up to 40 cycles, although one test piece cracked more rapidly than the others and failed generally at 100 cycles. Many of the cracks were at a top fold of the M-shaped seal. These joints were originally 1 in. wide instead of ¾ in. as specified.

J-4. This seal was similar to the seal on the J-7 joint, and failed in a similar way. Although the joint seemed tight and did not leak mortar, it would not close quite the full ¾ in. The first cracks were at the corner junction and along the sharp folds of the M-shaped section. Two types of end seal were provided — one of copper, which cracked even more rapidly than the top seal, and the other a 1/16-in. galvanized steel plate across the end of the air chamber. This device was fairly satisfactory, although it bent slightly in the repeated tests. The cracks in these joints at 40 cycles were 2 to 4½ in. long.

J-2. The nominal width of this joint was 1 in. The bottom of the steel air cell was notched near the ends to allow room for the end seal, and this notch allowed some mortar to seep into the air chamber, but due to the 1-in. original width, the joint could still be closed a full ¾ in. The soldered seams of the mitered corners of the joint opened at the first cycle of loading and cracks formed quickly in the trough of the seal at the corner, then spread along both the top and end seals. Several cracks developed along edges of the seal where the copper was bent through 180 deg. around a wire to form a bead. Failure of these copper seals was generally complete at 100 cycles of test.

J-5. This copper seal, which like that on the J-4 joint was of the M-type, failed completely within 60 cycles of test. The principal cracks occurred at the junction between seal and concrete slab. Apparently the seal was made of a hard grade of copper or was bent too sharply at the junction noted above, judging from the speed with which the failures developed.

J-3. This joint had a copper seal of the M-type. The construction of the joint permitted some mortar to leak into the air chamber during concreting. Generally the cracks in the copper were small during the first 40 cycles of test; however, they developed to complete failure during the next 100 cycles. The failures were similar to those of the J-4 joints, with cracks running along the top folds of the M-section.

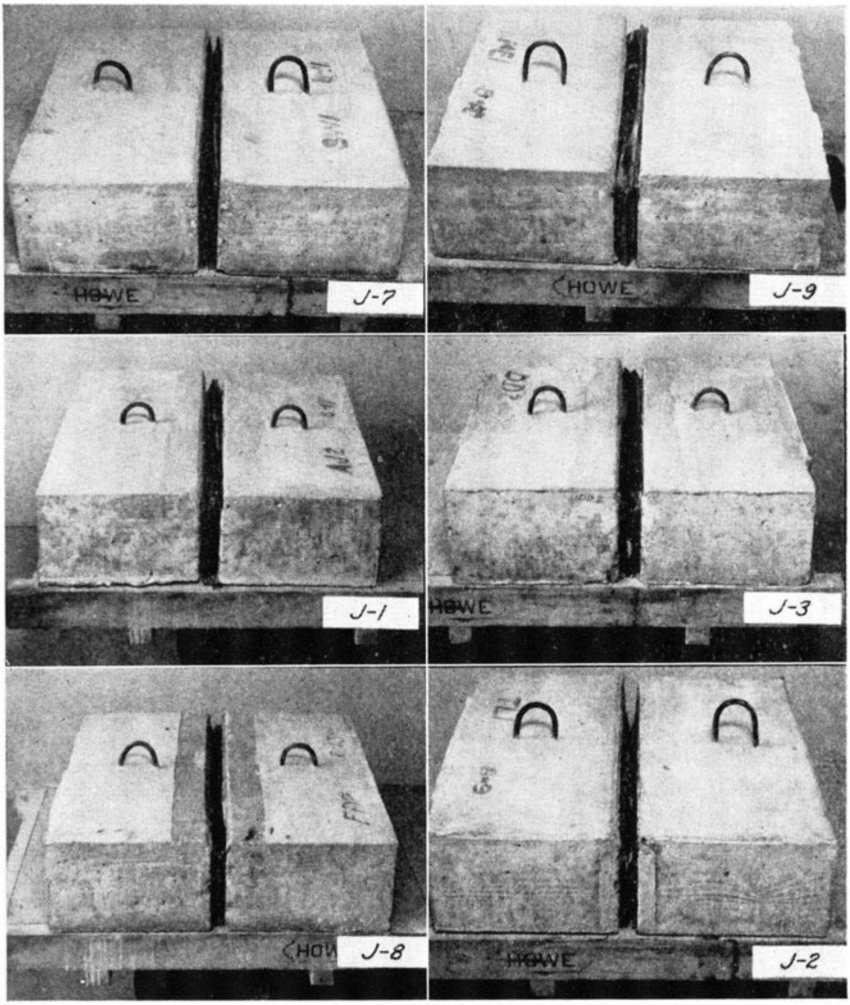


FIG. 57. TYPICAL JOINTS AFTER OPENING-CLOSING TESTS (UNIVERSITY OF ILLINOIS TESTS)

Views of a few of the previously discussed test pieces after failure of the copper seals are given in Fig. 57.

The conclusion from these tests is that no joint behaved satisfactorily. None withstood 40 cycles of opening and closing through a full ¾ in. of movement, without some damage to the seal. Of the joints that permitted ¾-in. closure, the J-1 had the fewest and smallest cracks at 40 and 100 cycles, and withstood the most cycles of loading without general failure. The J-3 and J-7 joints may be rated next, followed by the J-2, J-4, and J-5. The J-6, with only 7/16-in. closure, withstood the test very well and showed only one small crack in three test pieces at 40 cycles. The smaller movement was evidently much less severe than the full 3/4 in. The seal on this joint has a good feature in that neither closure nor the pounding of traffic on the asphalt cap tends to produce a tight fold in the seal, which is the usual source of cracking.

Of the three joints with preformed fillers, the J-10, with about 5/16-in. closure, made an excellent showing, with no cracks at 100 cycles and relatively little damage at 160 to 285 cycles. The fiber filler protected the copper seal from being folded tightly and the small amount of closure did not produce severe bending of the copper. The seal on the J-8 joint had a large number of small cracks at 40 cycles of about 7/16-in. closure, and the seal on the J-9 joint had several long cracks at 40 cycles of less than 1/2-in. closure.

The wide variation in results from the opening-closing tests indicates that while no joint gave the desired performance, some had been developed much further than others. Further improvement may be possible, but the known low fatigue resistance of copper and the large movement demanded in the joint are not encouraging. As indicated by the J-6 and J-10 joints, it would be far simpler to design a seal for a 1/4- or 3/8-in. opening than for the 3/4-in. movement specified. The use of a filler material in such a way as to prevent tight folds in the seal also seems highly desirable.

The tests gave no indication of the effect of vertical traffic loads crossing the joint continually and the large number of small daily movements due to temperature changes, factors which undoubtedly contribute greatly to the failure of the copper seals in service.

(g) Studies of Resistance of Joints to Collapse and Leakage during Installation

This small group of tests, made only on joints of the air-chamber types, was planned to determine how well the steel air chamber maintained its original width during installation, when subjected to pressure from fresh concrete similar to that produced by concrete pushed ahead of the strike-off screed of a finishing machine. The test also showed any leakage of fresh concrete or mortar into the air chamber.

A 5-ft. length of joint was placed in a wood form and held in position at each end by a small length of steel angle attached to the form. In making the test, three batches of 1:2:4 concrete with about 2-in. slump were used. The first batch was piled against the sides of the joint and rodded vigorously, care being taken to avoid striking the joint with the tamping bar. The other two batches were placed in the form and levelled and compacted by use of a Jackson surface vibrator. A movable bulkhead at one end of the form was then moved toward the joint a distance of 3 in. by means of two screw jacks. The pressure produced by this movement was not transmitted to the joint to any great extent, since the concrete relieved itself by bulging upward at the top surface. The surface was again vibrated, levelled, and finished with a trowel. Since the forms were 8 in. deep, a 1-in. wood filler was used over the top of the 7-in. joint, and the forms were filled the full 8 in. depth.

After 24 hr., the two slabs were jacked apart and the joint was opened up for examination and measurement. The departure of the two sheet steel walls of the air chamber from a vertical plane through the joint was measured at 75 points, 1 in. apart vertically and 4 in. horizontally. These departures furnished the information concerning widths of air chamber for the several joints shown in Table 10.

The measured widths listed in Table 10 do not give so complete information as desired, since it was impossible to get accurate measurements of the air chambers prior to the collapsing tests. Furthermore, none of the joints as submitted was uniform or true to design dimensions. The J-2 and J-3 joints were originally a full inch or more in width, and the J-4, J-5, and J-9 joints were evidently between 7/8 and 1 in. in width. The J-1 joints were fairly close to the designed 3/4 in. in width, while the J-6 joints were originally about 9/16 in. wide. The average widths after test show comparatively little decrease in width because of the concrete pressure, but the minimum widths indicate that at some point in the joint, the width of the air chamber was pretty generally 0.10 to 0.15 in. less than the average width, thus reducing by this amount the effective expansion space. Most of the extreme variations, found near the edges of the joint, were probably no greater than would be found in the joints in service installations.

The leakage of mortar into certain of the air chambers was noted in the discussion of the opening-closing tests. Here again a considerable amount of leakage occurred in the cases of the J-3, J-6, and J-7 joints.

TABLE 10
MEASURED WIDTHS OF AIR CHAMBERS OF EXPANSION JOINTS AFTER INSTALLATION TESTS
(University of Illinois Tests)

Dimensions in.	Joint						
	J-1	J-5	J-7	J-2	J-6	J-4	J-3
Average width......	0.71	0.92	0.93	0.97	0.55	0.88	1.05
Maximum width....	0.97	1.00	1.03	1.03	0.72	0.97	1.15
Minimum width....	0.60	0.84	0.78	0.87	0.41	0.75	0.87

The mortar filled the bottom of the air chamber to a depth of ¼ to ½ in., which would, to some extent, obstruct the closing of such a joint in service.

It may be concluded from these limited tests that: (1) some joints will permit an undesirable amount of concrete or mortar to leak into the air chamber, thus forming an obstruction to complete closure; and (2) the pressure exerted on the joint walls during installation produces an appreciable variation in joint width, so that the full designed provision for expansion may not be secured.

(h) Compression Tests of Joints and Joint Fillers

Compression tests were made on concrete specimens containing samples of the complete joint, in order to throw light on the ability of pavement edges to resist the forces which are present when the joint is under compression due to expansion of the concrete. Tests were made also to determine the resistance to compression of the premolded joint fillers which were a component part of some of the joints. These tests are discussed below.

(1) Tests of Complete Joints. The test specimens consisted of a 12-in. length of joint for a 7-in. pavement, cast midway between the ends of a concrete slab 7 in. thick, 12 in. wide, and 17 in. long. Figure 58 shows a specimen set up in the testing machine. The specimens were tested at the age of 14 days.

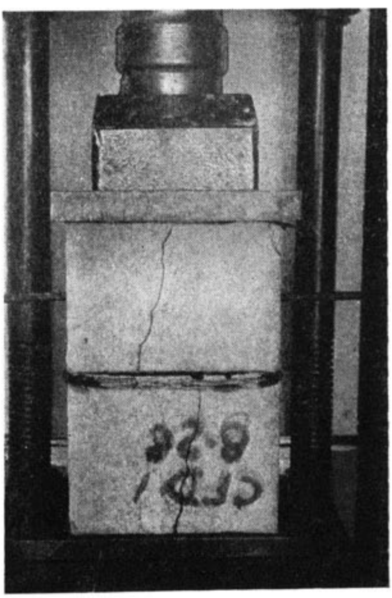


Fig. 58. Arrangement of Compression Test (University of Illinois Tests)

The joint was closed slowly by applying an axial load to the test specimen, the load and amount of closure of the joint being noted at initial failure and at complete failure. The results of these tests are given in Table 11. It is evident from the tests of all of the joints that there was uneven bearing along the face of the joint, resulting in high concentrated forces which split the concrete at compressive loads much lower than would have been predicted from the compressive strength of the concrete used in these specimens. This was particularly true in the case of the air-chamber joints, where small variations in the width of the joint and the metal separators between the walls of the joint limited the area over which the load on the specimen was distributed. The joints with premolded fillers and those in which premolded asphalt blocks were used as spacers were not so susceptible to these concentrated loads, and withstood considerably more compression before failure occurred. It appears that the premolded fillers and spacers served to distribute the compressive stress more uniformly over the face of the joint.

It may be noted that the fiber fillers used in the J-8, J-9, and J-10 joints showed very little extrusion at the maximum loads, but the sandwich type filler of the L-15 joint and the fibrated asphalt filler used in the J-11 "non-extruding" type joint extruded a large amount in this test, as indicated in Table 11.

The average strength of the concrete used was slightly more than 4,000 lb. per sq. in. Thus, a prism of solid concrete with a section of 84 sq. in. would carry more than 300,000 lb. However, in only one case did any of the test specimens carry more than 200,000 lb. (the L-15 specimens carried an average load of 241,800 lb.). It may be inferred, therefore, that a slab containing an expansion joint will be considerably weaker in resistance to longitudinal crushing than a plain slab. This is due to localized compressive stresses produced by the joint and load transmission devices, and to planes of weakness produced by the copper seals and anchors. In defense of the joints it may be said that blowups in plain concrete pavements will also be produced by stresses that are localized or eccentrically applied. However, the relatively low strength of many of these joints in compression, and the variation between the different types, merit consideration in any study of the properties of such joints.

(2) Tests of Asphalt Impregnated Fiber and Premolded Asphalt Fillers. Three of the joints submitted contained fiber-board fillers impregnated with asphalt, and one premolded asphalt filler was also submitted. The J-9 joint contained a ¾-in. fiber-board filler. The J-10 joint utilized a ½-in. fiber filler, while the J-8 joint contained an unidentified ¾-in. filler.

When these tests were made, no specifications for the material were available to the writer, so that the tests devised may not conform very closely to the tests now in general use. In addition to tests to determine the bitumen content of the fillers, two sets of compression tests were made.

The first compression tests were made on samples about 6 in. wide and 12 in. long, using the test arrangement shown in Fig. 59. The filler was com-

TABLE 11
RESULTS OF COMPRESSION TESTS OF JOINTS
(University of Illinois Tests)

Kind of Joint	Specimen	Load, lb.		Approximate Width of Joints, in.			Manner of Failure
		At initial failure	Maximum	Original	At initial failure	At failure	
J-1	1	36,000	91,100	3/4	3/16	0	Vertical cracking—lower slab.
	2	64,300	119,500	7/8	3/16	0	Vertical cracking—lower slab.
	3	46,800	106,600	3/4	1/4	0	Vertical cracking—top slab.
			105,700				
J-4	1	49,300	163,500	7/8	1/8	0	Failed along seal and dowel.
	2	22,000	113,900	7/8	11/32	0	Cracked along dowel and seal.
	3	40,100	99,400	7/8	1/4	3/16	Vertical cracks, lower slab.
			125,600				
J-5	1	47,300	111,700	13/16	5/32	0	Vertical cracks and along seal.
	2	61,100	99,800	3/4	1/8	0	Splitting of both slabs.
	3	41,800	99,300	7/8	3/16	0	Vertical cracks, upper slab.
			103,600				
J-6	1	28,500	123,000	1/2	3/8	1/32	Failed at corner—local bearing.
	2	21,000	134,500	1/2	1/4	0	Spalling and cracking at low loads.
	3	23,000	142,900	1/2	1/32	0	Spalling and cracking at low loads.
			133,500				
J-2	1	28,600	185,900	31/32	5/16	0	Diagonal cracks in top slab.
	2	61,900	176,900	1	1/4	0	Vertical cracking and along seal.
	3	22,300	186,000	31/32	11/32	0	Initial cracks in plane of seal.
			182,900				
J-7	1	17,800	70,800	1	5/8	3/8	Vertical cracking.
	2	95,000	179,000	1	3/16	1/8	Vertical cracking.
	3	70,100	160,000	1 1/32	5/16	1/4	Vertical cracking—cracks along seal.
			136,600				
J-3	1	48,900	147,200	1	3/8	3/16	Cracks along seal—vertical cracks.
	2	58,700	90,000	1	3/8	1/4	Splitting and cracks along seal.
	3	52,600	192,900	1	3/8	1/8	Split along seal and across face.
			143,400				
J-10	1	87,200	189,300	1/2	1/4	7/32	Cracks along L-6 plates.
	2	49,100	153,050	1/2	1/4	1/4	Vertical cracks along L-6 plates.
	3	198,500	219,600	1/2	1/4	1/4	Vertical cracking at several points.
			187,300				
J-8	1	37,700	163,300	3/4	15/32	3/8	Vertical cracking.
	2	128,300	156,500	13/16	3/8	3/8	Cracks along seal and edges of slab.
	3	66,700	114,700	3/4	3/8	11/32	Vertical cracks along seal.
			143,600				
J-9	1	48,700	169,200	3/4	11/32	3/16	Cracks along edges of slab.
	2	90,600	144,200	3/4	11/32	5/16	Cracks along edges of slab.
	3	39,300	151,500	3/4	11/32	5/16	Cracks along edges of slab.
			155,000				
L-15	1	235,000	235,000	0.3	..	0	(Extrusion began at 50,000 lb. load,
	2	225,400	225,400	0.3	..	0	extended 2 in. at 55,000 lb., and
	3	265,000	265,000	0.3	..	0	6 in. at failure.)
			241,800				
J-11	1	79,000	79,000	1.0	..	0.63	(Extrusion began at 10,000 to 15,000
	2	70,000	88,300	1.0	..	0.46	lb. load; final extrusion varied
	3	90,000	90,000	1.0	..	0.43	from 3/4 to 1 3/4 in.)
			85,800				

Cross section of slab 7 in. by 12 in. Average concrete strength, 4,090 lb. per sq. in.

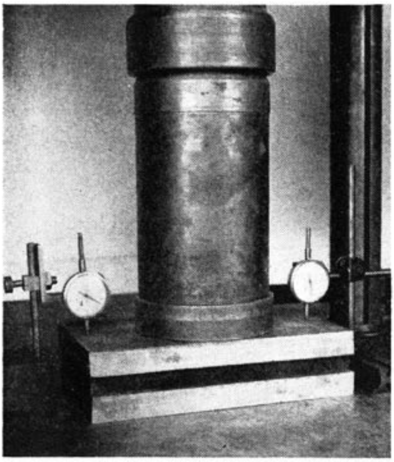


FIG. 59.
ARRANGEMENT OF TEST OF FIBER AND ASPHALT FILLERS
(UNIVERSITY OF ILLINOIS)

TABLE 12
RESULTS OF COMPRESSION TESTS OF FIBER AND ASPHALT FILLERS
(University of Illinois Tests)

Item	Type of Joint Used with				
	J-9[1]	J-9[1]	J-10	J-8	J-11
Bitumen content (per cent)	29.1	34.4	48.7	45.6	.....
Width, in.	5.87	5.62	6.5	5.62	7.0
Length, in.	12.0	11.5	12.0	12.0	12.0
Thickness, in.					
1. Original	0.75	0.75	0.51	0.75	1.00 channeled
2. At 250,000-lb. total load	0.210	0.207	0.175	0.282	0.20
3. With load removed	0.241	0.258	0.224	0.306	No recovery
4. After 5 min.	0.265	0.305	0.260	0.337	
5. After 24 hr.	0.295	0.420	.....	.....	
Extrusion—increase in width, in.	0.06	0.06	0.09	1.62	3.0 to 6.0

[1] Material from two manufacturers submitted.

pressed between rigid steel plates and the change in thickness was measured by means of two micrometer dials, reading to 0.001 in. The tests were carried to a load of 250,000 lb., or about 3,500 lb. per sq. in. on the specimens. The load was then removed and the thickness determined. The recovery after 5 min. and after 24 hr. was also noted. Table 12 summarizes the test results.

For the fiber fillers, the load required to compress the material to half its original thickness was not very great, varying from about 12,000 to 25,000 lb. total load, or 170 to 350 lb. per sq. in. Beyond this point the resistance of the material increased rapidly. The maximum applied load of 250,000 lb. produced a total reduction in thickness of 63 to 72 per cent, based on the original thickness of the specimen. All of these fillers showed a considerable recovery in thickness when the load was removed, even though the compression had been severe. The recovery of ¾-in. fiber specimens immediately after release of the maximum load was 0.03 to 0.05 in.; after 5 min. it had increased to 0.05 to 0.10 in., and after 24 hr. the total recovery was 0.10 to 0.21 in.

The extrusion of the fiber samples was generally about 1/16 in. in a 6-in. width at the 250,000-lb. load. The filler used in the J-8 joint, however, extruded badly, especially at the middle of the sides of the specimen, where the increase in width of the sample was as much as 1⅝ in.

The behavior of the fibrated asphalt filler used with the "non-extruding" type joint differed from that of the fiber fillers. The test samples, about 7 in. by 12 in., contained two channels or grooves on each side, 1 in. and 1½ in. wide, respectively, and ¼ in. deep, reducing the normal thickness of 1 in. to ½ in. at these grooves. In the test, the material flowed under the applied load, so that the grooves disappeared at a load of about 38,000 lb., when the thickness of the material was about 0.72 in. As the load was increased to 250,000 lb., the thickness decreased at a fairly uniform rate to a final value of 0.20 in. By this time the filler had extruded in a thin sheet from all sides of the samples to a distance of 3 to 6 in. outside its original position. As might be expected, the material showed no recovery or return toward its original thickness when the load was removed.

A second group of tests on the fillers was made in which enough load was applied to reduce the filler thickness 50 per cent. Such loads were applied and removed five times. The results are given in Table 13. It is seen that the required load became smaller at each application, due to yielding of the material. Extrusion of the fiber material was negligible; that of the J-8 joint was about 3/16 in. at the widest part of the specimen.

These tests confirm manufacturers' statements that fiber fillers can be provided which will have very little extrusion. The rapid increase in resistance to compression after the board has been compressed 50 per cent gives a very good idea of the forces required to compress such a joint in a pavement, and indicates clearly that joints with fiber fillers allow only a limited width for pavement expansion. The data on recovery of filler thickness after the removal of the load, while unfortunately very incomplete, show much less complete recovery than manufacturers usually claim.

(i) Summary

The air-chamber expansion joint has four principal objects: (1) to provide a space (¾ in., every 90 ft., under 1937 Illinois specifications) for the pavement to occupy due to expansion and growth; (2) to keep this expansion space free from dirt, gravel, and other foreign material by means of a copper seal; (3) to provide means of transferring load across the joint, in order to preserve a smooth riding surface and to reduce stresses in the slab; and (4) to prevent surface water entering the subgrade through the expansion space.

TABLE 13

RESULTS OF REPEATED COMPRESSION TESTS OF FIBER FILLERS
(University of Illinois Tests)

Joint Filler	J-9		J-10		J-10		J-8		J-8	
Width, in.	5⅞		6½		6 7/16		5 9/16		5⅝	
Length, in.	12		12		11⅞		11 13/16		12	
Thickness, in.	0.77		0.51		0.52		0.75		0.76	
	Load lb.	Thickness in.	Load lb.	Thickness in.	Load lb.	Thickness in.	Load lb.	Thickness in.	Load lb.	Thickness in.
Load Off	0	0.770	0	0.510	0	0.520	0	0.750	0	0.760
On[1]	16,330	0.390	28,270	0.256	28,930	0.258	39,720	0.363	34,200	0.379
Off	0	0.473	0	0.336	0	0.338	0	0.416	0	0.454
On	13,520	0.389	21,390	0.260	22,520	0.260	25,700	0.363	28,200	0.378
Off	0	0.460	0	0.329	0	0.331	0	0.409	0	0.411
On	12,470	0.390	21,430	0.259	21,750	0,260	29,300	0.357	27,800	0.379
Off	0	0.458	0	0.326	0	0.326	0	0.405	0	0.456
On	11,720	0.390	20,100	0.260	21,750	0.260	15,600	0.366	30,100	0.371
Off	0	0.454	0	0.323	0	0.323	0	0.405	0	0.448
On	11,660	0.390	20,150	0.260	20,800	0.261	17,450	0.366	23,300	0.378
Off	0	0.451	0	0.322	0	0.321	0	0.408	0	0.434
After 1 hr. Off	0	0.530	0	0.360	0	0.360	......	.....	0	0.497

[1] The loads applied in these tests were those required to compress the fiber board to one-half of its original thickness.

Tests were made on eleven complete expansion joint assemblies, and on additional load transmission devices and joint fillers. The tests covered: (1) the strength and effectiveness of load transmission devices; (2) the anchorage of copper seals into the adjoining concrete slabs; (3) the resistance of the copper seals to fatigue due to repeated opening and closing of the joint; (4) studies of the tendency of joints to collapse and of leakage of concrete into the air chamber during installation; and (5) compression tests of all joints, such as might occur with complete closure of the joint. These tests were not expected to duplicate the effects of traffic and weather on a joint in service; at best they formed a group of simple, comparative tests which determined some of the important properties of the joints, but could not possibly duplicate many of the effects of conditions in service.

It is difficult to make an accurate comparison of the various joints as a result of the tests, because out of the group of ten complete joints, only six had air chambers ¾ in. or more in width, as requested for these tests (three of these were 1 in. wide, but contained soft asphalt spacing blocks). One of the others had a ½-in. air chamber; the remainder, ⅜, ½, or ¾-in. asphalt-impregnated fiber fillers. In the load transmission test it is obviously easier to span a ½-in. gap than a ¾ or 1-in. gap; hence the test values give the narrow joints an advantage they probably would not enjoy if all joint widths were equal. Similarly, in the opening-closing tests, the full closure of a ¾-in. air-chamber joint was a far more severe treatment than the partial closure of the fiber-filled joints, which could not be closed to less than 5⁄16- to ½-in. width.

It is probably impossible to give ratings to the various joints on the basis of the laboratory tests. A brief summary, however, of the performance of the various joints in the load transmission tests, and in the fatigue tests of copper seals, gives information on the most essential features of the joints (Table 14). The joints are divided into two groups, those with air chambers ¾ in. or more in width and those with fiber fillers or narrower widths. It is evident that a joint may have some excellent properties, but be sadly lacking in others. Regarding the load transmission tests, it is important to note that the comparisons given are based on equal spacings of the several dowel units, since the test loads given are per linear foot of joint, or per device at 12-in. spacing in the test piece. If any of the devices are used at wider spacings, this will modify the computed effectiveness shown.

In summary, the following statements may be made concerning the individual joints, based mainly on the test results.

TABLE 14

SUMMARY OF LOAD TRANSFER AND OPENING-CLOSING TESTS

(University of Illinois Tests)

Load Transmission Test					Opening-Closing Tests			
Joint designation	Joint width in.	Load transmission device	Maximum load lb. per lin. ft.	Effectiveness, 2,500 lb. per lin. ft. medium soil	Joint designation	Joint closure in.	Notes on fatigue resistance of copper seals, opening-closing tests: Cracks at 40 cycles	Manner of failure
GROUP I. JOINTS WITH ¾- OR 1-IN. AIR CHAMBERS								
J-1	¾	L-1	12,205	0.77	J-1	¾	4 to 8, ¼ to ⅝ in. long	Cracks 2½ to 4½-in. at 200-360 cycles
J-5	¾	L-1	12,175	0.75	J-3	¾	4 to 7, ⅛ to 1½ in. long	Gen. failure at about 150 cycles
J-7	1	L-10	10,798	0.70	J-7	¾	5 to 11, ½ to 3 in. long	Gen. failure at 100 to 164 cycles
J-4	¾	¾-in. dowel	7,020	0.78	J-4	¾	5 to 7, 2 to 4½ in. long	Many large cracks at 115 to 170 cycles
J-3	1	L-5	8,625	0.55	J-2	¾	8 to 14, 3 to 6 in. long	Gen. failure at 87 to 141 cycles
J-2	1	L-2	4,090	0.51	J-5	¾	9 to 16, ½ to 9 in. long	Gen. failure at 52 to 61 cycles
GROUP II. JOINTS WITH ½-IN. AIR CHAMBER OR FIBER FILLER								
J-9	¾	1-in. dowel	12,930	0.83	J-10	5/16	No cracks	Few cracks at 160 to 285 cycles
J-6	½	L-4	12,595	0.89	J-6	7/16	1 crack, ¾ in. long	Cracks ⅜ to ⅞ in. at 100 cycles
J-8	¾	¾-in. dowel	7,670	0.77	J-8	7/16	4 to 7 cracks, ¾ in. long	Gen. failure at 150 cycles
J-10	½	L-6 (3/16-in. x 1½-in. plate)	5,475	0.49	J-9	½	4 cracks, 8 to 15 in. long	Gen. failure at 47 to 102 cycles
J-15	⅜	L-15	(4,270 5,860	0.87) 0.57)	...	..		

J-1. The copper seal on this joint appeared to be one of the best submitted for a ¾-in. joint. The air chamber was tight but had variations in width. The L-1 load transmission unit developed high shear properties and fairly effective load transfer. The copper seal seemed well anchored, although field inspection showed some honeycombed concrete under seals, also splitting of seals under traffic.

J-5. The copper seal on this joint appeared to be very susceptible to splitting and early failure. The seal could be well anchored, but provided a good chance for honeycombed concrete to occur. The air chamber was quite uniform in width and resistant to collapse. Tests of the L-1 load transmission device used with this joint verified the results obtained from tests of the J-1 joint.

J-7. This copper seal, like the J-5, was of the M-type, but it showed better resistance to flexure than the J-5. The asphalt spacing blocks in the air chamber offered resistance, but permitted full closure with reasonable pressure. Mortar leaked into the air chamber. The L-10 dowel units used had fairly high shear properties and effectiveness, considering their use on a 1-in. joint width. The anchorage of copper seals was only average; it would probably produce honeycombing and spalling in service.

J-4. This copper seal, of the M-type, failed fairly early under repeated flexing. Splitting along one of the top folds was prevalent. The conventional dowel bars gave good service, but their load deflection characteristics were not so good as those of the L-1 device. Anchorage of the seals seemed good, but the overhanging wings would probably cause honeycombing and spalling of concrete.

J-3. These M-type copper seals appeared about as good as the J-7 seals in fatigue resistance, and better than the J-4 and J-5 seals. The L-5 unit was somewhat stronger in shear, but more flexible than the conventional ¾-in. dowel bar. Like the J-7 joint, the construction of this joint with asphalt filler blocks caused some resistance to closure; there was also leakage of mortar into the air chamber.

J-2. The load transmission devices, which were an integral part of this joint, carried relatively low shear and showed large deflections, resulting in low effectiveness of load transfer. The copper seals failed rather quickly under repeated flexing. The anchorage of the seals was fair. The construction permitted leakage of mortar into the ends of the air chamber, and the asphalt spacers introduced some resistance to closure.

J-6. This joint, with a ½-in. air chamber, showed high shear properties and effectiveness of its load transmission devices, and fairly good resistance of the copper seal to repeated flexing. The anchorage of the seals was good, and their design would probably result in fairly good bonding in actual road construction. The joint permitted some seepage of concrete into the air chamber during concreting. There was some breakage of the gray iron dowel castings in handling, also at failure in the load transmission test. The soft rubber fillers used outside the copper seal at the ends of the joint appeared to be of doubtful permanence.

J-9. The 1-in. dowel bars used in this joint produced high shear properties and rigidity in the load transmission test. This was also true of oval pipe dowels furnished as an alternate load transmission device. Analysis indicates that such dowels might be so stiff as to cause trouble when warping of slabs is present. The copper seal was not effective in resisting repeated flexing, and the friction tape provided for making the corner seals water-tight came loose at the first closure of the joint. The narrow wings of the copper seal gave good anchorage. The ¾-in. fiber joint filler prevented full ¾-in. closure of the joint. The fabrication of this joint was such as to make it very heavy and awkward for field installation.

J-10. The load transmission device in this joint had low shear properties and stiffness. The 1½-in. plate dowel, evidently too narrow, promoted early splitting of the adjacent concrete slab. The projecting edges of the copper seal were not wide enough to secure adequate anchorage in the concrete. However, the copper seal was very resistant to repeated flexure, perhaps because it did not receive such severe punishment as most of the other seals, the ½-in. fiber filler permitting only about 5/16-in. closure of the joint. This type of seal, enclosing a filler which prevents tight folds in the copper, showed good fatigue resistance.

J-8. The conventional ¾-in. dowel bar used in this joint gave slightly better results than that used in the J-4 joint. The wire dowel supports are a doubtful feature, since to be effective the dowels must be placed and maintained parallel to each other with a high degree of accuracy. The wires appeared to be easy to displace with a shovel, vibrator, or tamping tool. The copper seal gave low resistance to repeated flexing; apparently it acted satisfactorily until failure of the sheet steel liner inside the copper, then failed rapidly. The anchorage of the copper seals seemed to be adequate. The ¾-in. fiber filler, as in the J-9 joint, prevented more than 7/16-in. closure. The joint might be hard to handle and to align during construction.

L-15 Load Transmission Device. This joint had no copper seal, and no air chamber. The bituminous premolded filler permitted a closure of about 0.3 in. when practically all of the asphalt had been extruded from the joint. The joint was stronger in shear, but more flexible, when the load transmission plates were in tension than when they were in compression. The results of the compression test indicated that this joint did not produce highly localized stress. The 14-gage steel plates which formed the load transmission members are very thin for use where exposed to moisture and resulting corrosion.

L-11 Load Transmission Device. This unit, submitted without an accompanying expansion joint, developed a shearing strength intermediate between the conventional ¾-in. dowel bar and the L-1 unit. It exhibited high stiffness and effectiveness in load transfer. The manufacturer stated that both these units and the L-10 units were inadvertently furnished with dowel pins of soft steel. It is probable that the unit could be improved by using pins of stronger steel and by increasing the length of the wing anchors.

L-16 Load Transmission Device. These dowels showed greater shearing strength in the load transmission test than those of any of the other devices tested. However, as tested, they were able to transfer load in only one direction across a joint. Hence, in service two dowels, faced in opposite directions, would be required to be equivalent to one dowel of the ordinary bar type. On this basis, the L-16 dowel is but slightly stronger in shear and of about the same effectiveness as a conventional ¾-in. dowel bar. Such effectiveness is dependent upon accurate alignment of the bearing faces of the device during installation. The device is about 50 per cent heavier than the conventional ¾-in. dowel bar.

The foregoing discussions indicate that most of the fabricated joints had some very unsatisfactory features. The repeated opening-closing tests of the copper seals, supplemented by observation of certain of the seals in highway service, indicate that none of the copper seals offers any assurance of lasting as long as the concrete pavement, particularly if the seal must be designed to provide for a ¾-in. movement. Even if the seals remain unbroken, water and dirt can enter the joint from the subgrade. A satisfactory corner joint between top and end seals is another detail of these joints that is still to be developed. Since the copper seals are a vital and expensive feature of the air-chamber joints, their performance should be practically perfect to warrant their use.

Several of the load transmission devices appear to give satisfactory shearing strength and effectiveness, when carefully installed. This series of tests should indicate the deficiencies and defects to be corrected in some of these devices. However, it seems probable that the record of their performance in highway service will be the best indication of the usefulness of such units.

The use of mesh reinforcement, although touched upon very briefly in these tests, seems promising as a means of crack control and reduction in faulting and poor riding qualities of highway slabs.

10. *Illinois Division of Highways Tests.* — When metal expansion and contraction joints were adopted for use in Illinois pavements, engineers in the Division of Highways were faced with an entirely new problem — testing the joints submitted for approval to determine whether they were suitable and satisfactory and could be expected under the anticipated conditions of service to remain serviceable and operative for the economic life of the pavement. There were no established methods of test or procedures to follow, no previous experience to serve as a guide. It was necessary not only to anticipate service conditions but to devise tests which, in a short time, would produce the results to be expected from years of actual service. Furthermore, new testing equipment and machinery had to be designed and built.

It was natural, therefore, that the joint testing program of the Illinois Division of Highways, as in the case of any pioneer work, progressed through a period of development during which changes were made as dictated by experience. Much of this development work will not be reported here because its only value was that it furnished knowledge from which standard methods of tests were finally evolved. Most of the joints and load transmission devices considered by the Illinois Division of Highways were tested in accordance with methods sufficiently standardized to make the results reasonably comparable.

(a) Outline of Laboratory Tests

It was the function of the tests to subject samples of joints to conditions as nearly similar to those to be anticipated in actual service as could be produced in the laboratory, and in such a manner that results could be obtained within a reasonably short time which would indicate with assurance how well the joint might be expected to meet requirements of actual service. While it was impossible to duplicate all of the destructive forces a joint must resist in actual service, it was possible in most cases to devise tests which would establish the relative behavior of different joints and furnish information which, when properly analyzed, would permit an estimate of the ability of a joint to stand up under service requirements.

In general, the program included a test to determine the ability of the joint, or some auxiliary device incorporated in it, to transfer load from one slab to another; a study of the ability of the copper seal to withstand the fatigue caused by the constant flexing produced by expansion, contraction, and warping of the concrete; a study of the durability of the anchorage of the copper seal in the concrete; a study of the resistance of air chamber joints to collapse and deformation caused by pressures produced by placing and compacting fresh concrete around them; and a study of the compressibility of the joints and joint material.

(1) Load Transmission Test. This test is in effect a calibration of the member whose purpose it is to provide mutual support between slabs, to determine the relation between shear and deflection; in other words, to determine the amount of load transferred for various amounts of deflection across the joint. The test specimens, 7 in. deep, 10 or 12 in. wide, and 38 in. long, were divided into three sections by two pieces of joint containing the load transmission feature or device, placed approximately at the one-third points of the specimen. When the load transmission feature was an integral part of the joint, sections of the joint were used to divide the specimen; when the load transmission device was a separate unit, sections of the joint or pieces of bituminous premolded fiber board were used. The width of joint was generally ¾ in., but in several instances ½ in. or 1 in. Wherever possible, the devices were placed with their axes coincident with the longitudinal axis of the specimen, although in a few cases it was necessary to stagger the two units because of their length.

The concrete mixture generally used was designed for a compressive strength of 5,000 lb. per sq. in. at seven days, and the unit proportions were 205 lb. of sand, 343 lb. of 1-in. gravel, and 5.4 gal. of water per bag of high-early-strength cement. Extreme care was taken to assure dense concrete against the joint and around the load transmission devices. The concrete speci-

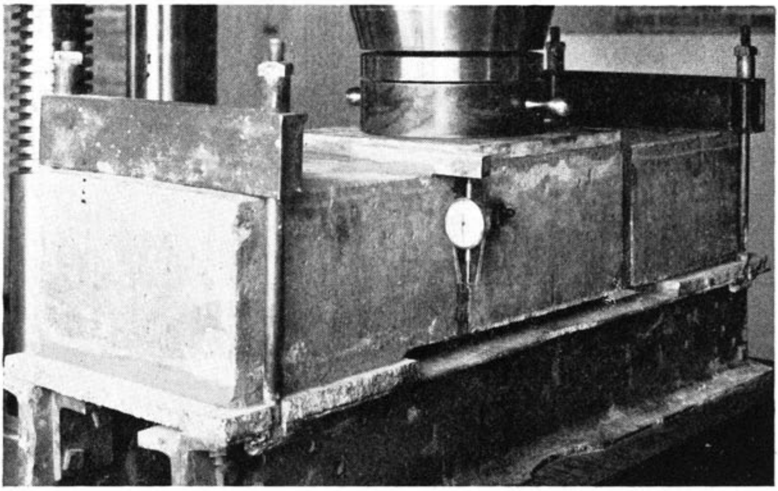


FIG. 60. LOAD TRANSMISSION TEST SPECIMEN SET UP IN MACHINE READY FOR TESTS (ILLINOIS DIVISION OF HIGHWAYS TESTS)

mens were cured at 70 deg. F. and 95 per cent relative humidity until tested. Usually the specimens were tested at the age of seven days.

In testing a specimen, its end sections were embedded in plaster of paris and portland cement mortar, on a test base which supported the end sections and left the center section completely suspended by the load transmission devices. In early tests, the support under the end sections extended from the end of the specimen to the joint; later the edge of the supporting plate was moved to a point 4 in. away from the joint to accommodate load transmission devices with wide bases. Loads of increasing magnitudes, usually in 2,000-lb. increments, were applied to the center section, through a ¾-in. steel plate bedded on the top of the section, until the specimen failed. Each load was applied and released five times before one of the next higher magnitude was applied. Bending of the devices across the joints under the various loads was determined by means of micrometer dials. The properties of most load transmission devices were determined on the basis of the results from at least six test specimens, and were compared with results obtained from tests of load transmission devices of an approved type. Figure 60 shows a specimen in the testing machine ready for test, and the method used to measure deflections across the joint.

(2) OPENING-CLOSING TESTS OF COPPER SEALS. This test was devised to determine the ability of the copper seals on expansion joints to withstand the repeated flexing produced by temperature changes in the concrete. In the test, a short section of joint 12 or 24 in. long, set in a concrete specimen, was subjected to alternate closing and opening until the seal had completely

failed or until sufficient data had been obtained to establish its ability to withstand this action. Early tests were made in a shaper, that being the only machine available which would give the required reciprocating motion. Later tests were made in a specially designed hydraulic testing machine which produces a slow, horizontal reciprocating motion and has an automatic reversing mechanism and an accurately adjustable stroke. This machine was also used in similar tests made by the University of Illinois.

The procedure consisted of closing and opening the joint repeatedly and observing at intervals the number and extent of failures in the copper seal. The joints were completely closed except in the case of bituminous filled joints, which were closed to the point where the resistance to compression equalled the power of the machine. The action on seals only partially closed was less severe than when they were completely closed, which should be given due consideration in analyzing the test data.

The test simulates, in an accelerated manner, the action of the joint during annual cycles of movement. The large number of daily cycles, which take place as pavement temperature changes from day to night, were disregarded because those movements are small and it was thought they would not contribute to failure. It now appears, however, that daily movements may contribute in a large measure to failures caused by flexing due to temperature changes, particularly after the first annual cycle has bent the seal into a tight crimp.

(3) Anchorage Tests of Copper Seal. The procedure used by the Division of Highways in making these tests was the same as the method used by the University of Illinois, described in sub-section 9(e) (see page 76), except that full depth specimens were not used. In the Division of Highways tests, the depth of the concrete specimen was made twice the actual distance from the surface of the pavement to the point where the flange of the copper seal entered the concrete.

(4) Installation Test. The procedure used by the Division of Highways in making these tests was the same as that used by the University of Illinois, described in sub-section 9(g) (see page 85).

(5) Compression Tests of Joints. In the case of metal expansion joints, the procedure generally consisted of applying to the concrete specimen containing the joint sufficient compressive load to close the joint completely. Observations were taken at intervals of the load and the amount of closure. The load was not continued beyond that necessary to close the joints, because it was desired to use the specimens in the opening-closing tests. In the case of the J-1 joint specimens, the joints were closed various amounts corresponding to the theoretical maximum closing computed for joints placed at various seasons of the year.

The procedure followed in testing joints with premolded fillers was quite similar to that used by the University of Illinois, a description of which is given in sub-section 9(h) (see page 87).

(b) List of Joints and Load Transmission Devices Tested

The following expansion joints were tested as indicated:

	Tests Conducted			
Type	Opening-Closing	Anchorage	Installation	Compression
J-1	x	x	x	x
J-10				x
J-8	x			x
J-6	x	x	x	x
J-2 (Air Chamber)	x	x		
J-2 (Bit. Filled)	x			x
J-4	x	x	x	x

Load transmission devices included in the program of tests were L-1, L-2, L-3, L-4, L-5, L-6, L-7, L-8, L-9, L-10, L-11, L-12, and the conventional dowel bar.

(c) Load Transmission Tests

When the continuity of a pavement slab is broken by a joint, two transverse edges are formed which, unless some means of mutual support is afforded, may fail under the heavy wheel loads passing over them. There are three reasons for this. First, the corners formed by the intersection of the expansion joint with the center joint and the edges of the pavement are points of weakness. Second, in the thickened edge pavements, which comprise a large part of the system of highways in Illinois, the free transverse edges formed by a joint lie mostly in the thinner central portion of the pavement and are weaker than the outer edge of the pavement. Third, the dynamic effect of a wheel load rolling across a joint and being applied suddenly to the adjacent edge may approach 1½ times that of the same load applied statically. A load transmission device, by affording mutual support between adjacent slabs, strengthens the edges and corners formed by a joint.

The extent to which a device may perform that function depends on the ability of the device to transmit shear across a joint, and since the deflection of a pavement slab with normal subgrade support, under even the heaviest wheel loads permitted by statute, is relatively small, it is essential that a device be capable of transmitting shear with little deflection. It is also necessary that the elastic limit of a device, that is, the load at which definite permanent deformations begin to occur in one or more of the component parts of the device, be sufficiently higher than the anticipated shear to provide a reasonable factor of safety. A device also should be designed so that it will not produce critical compressive stresses in the supporting concrete which will cause bearing failures, commonly called funneling, and thus destroy the ability of the device to transmit load.

The load transmission tests made by the Illinois Division of Highways, as has been stated heretofore, were in effect a calibration of the device to determine the relation between the shear transmitted and the deflection across the joint. From this calibration it is possible to determine by comparison the relative stiffness, yield, and stress-reducing properties of any load transmission device. By making certain assumptions regarding subgrade support and pavement thickness, the relative ability of a device to transmit load across a joint in a concrete pavement may be predicted.

(1) Data from Tests. The data obtained from the load transmission tests consisted of deflections and permanent sets corresponding to various amounts of shear. Generally, the values for deflection and permanent set were the average of five readings taken at each joint for each load, the load being expressed in pounds per device, assumed to be half the applied load, since the applied load was carried by two devices.

As stated heretofore, a change was made in the test procedure during the period over which the tests extended. In earlier tests, the end sections of the specimen were supported for their full length, while in the later tests, the supporting plates were placed approximately 4 in. from the joints so that the end sections were supported for only two-thirds their length. This change in procedure, made necessary by the design of some of the devices submitted for test, had some effect on the results. For that reason, the data obtained by the two methods of tests will be presented separately.

Table 15 gives the average deflections and permanent sets for those devices tested by the earlier method. Table 16 gives similar data for the devices tested by the later method. In all cases the values are the averages obtained from tests of the number of specimens indicated in the table.

(2) Discussion of Test Results. In analyzing the test results, load deflection and load permanent set curves for the individual specimens, and composite curves representing the average results obtained from tests of a number of similar specimens, are of definite advantage. The characteristics of a load transmission device can be reasonably well predicted from the shape and slope of the curves and the magnitudes of load and deflection at various points on the curves. Also, by treating the data analytically, mathematical relations can be derived which make it possible to predict, at least comparatively, the amount of load transfer to be expected from a device under certain assumed conditions.

The method of analysis may be best shown by an example. For this purpose, the average data obtained from all the tests of L-1 devices will be used,

TABLE 15
DEFLECTIONS AND PERMANENT SETS OBTAINED FROM LOAD TRANSMISSION TESTS MADE ON SPECIMENS WITH END SECTIONS FULLY SUPPORTED
(Illinois Division of Highways Tests)

Device	Number of Specimens Tested	Width of Joint in.	Deflections and Permanent Sets, 0.001 in.										
			Load on one joint or device, lb.										
			1,000	2,000	3,000	4,000	5,000	6,000	7,000	8,000	9,000	10,000	11,000
L-6[1]	6	½	4.93	9.32	13.65	17.84	22.12	27.29	36.10	.....	.....	.....	.....
			0.98	1.78	2.59	3.73	5.26	10.33	17.50	.....	.....	.....	.....
L-6[2]	6	½	6.00	11.90	16.70	21.60	26.30	38.90	.....	.....	.....	.....	.....
			.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
L-6[3]	6	½	15.50	30.20	44.20	57.30	(4,500) 75.1	.....	.....	.....	.....	.....	.....
			.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
L-5	6	1	6.32	10.39	14.09	17.88	21.88	26.22	30.80	36.24	43.28	53.34	64.10
			0.76	1.38	2.02	2.82	3.60	4.59	5.69	6.83	7.88	12.40	.....
L-4	6	¾	4.50	8.30	12.40	16.60	21.40	26.50	31.60	37.10	43.10	52.40	63.90
			.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
L-8	6	¾	16.70	24.10	30.80	37.60	44.50	52.60	.....	.....	.....	.....	.....
			7.20	11.00	13.90	16.70	19.50	23.10	.....	.....	.....	.....	.....
L-6 (3/16-in. x 2½-in. plate)	6	½	11.87	20.80	28.18	36.23	.....	.....	.....	.....	.....	.....	.....
			3.44	6.19	8.67	12.48	.....	.....	.....	.....	.....	.....	.....
L-6 (¼-in. x 3-in. plate)	6	1	21.90	37.08	48.66	.....	.....	.....	.....	.....	.....	.....	.....
			8.48	15.45	21.28	.....	.....	.....	.....	.....	.....	.....	.....
Conventional dowel[4]	6	½	2.12	4.04	5.87	8.38	11.68	15.58	19.81	25.11	32.56	.....	.....
			0.36	0.67	0.93	1.41	2.10	2.87	3.66	4.83	8.10	.....	.....
Conventional dowel[4]	12	¾	2.00	5.00	9.00	13.50	20.00	27.00	35.83	47.00	64.00	.....	.....
			.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
Conventional dowel[4]	6	1	4.58	9.40	14.72	20.62	27.90	38.34	62.90	.....	.....	.....	.....
			1.10	2.02	3.08	4.00	5.07	8.05	24.67	.....	.....	.....	.....

NOTE: Values in upper line in each section of table are deflections under load; those in lower line are permanent sets after removal of load.
[1] Modified plate dowel joint. Plate dowel was integral part of side walls of joint.
[2] Plate dowel joint with ¼-in. x 1½-in. plate. Tests made at normal joint width.
[3] Same specimens as (2). Tests made with joint opened 0.22 in. beyond normal to determine action when slabs are pulled apart by contraction of concrete.
[4] ¾-in. round hot rolled dowels, 24 in. long.

TABLE 16

DEFLECTIONS AND PERMANENT SETS OBTAINED FROM LOAD TRANSMISSION TESTS ON SPECIMENS WITH SUPPORT UNDER END SECTIONS BEGINNING 4 INCHES BACK OF JOINT

(Illinois Division of Highways Tests)

Device	Number of Specimens Tested	Width of Joint in.	Deflections and Permanent Sets, 0.001-in. Load on one joint or device, lb. 1,000	2,000	3,000	4,000	5,000	6,000	7,000	8,000	9,000	10,000	11,000	12,000
L-11	6	¾	9.37 2.33	15.95 4.03	20.59 5.11	24.29 6.10	28.64 7.04	32.43 7.93	36.77 8.92	41.32 10.28	47.23 11.93	54.48 14.64	64.48 18.68	73.02 22.82
L-7	6	¾	8.83 1.80	13.63 2.83	17.87 4.09	21.42 5.03	25.17 6.07	29.13 7.03	33.74 8.38	39.30 10.39	46.65 13.25	52.52 15.22	....	....
L-8 (9-in. dowel) 1940	7	¾	4.63 0.71	8.97 1.37	12.69 2.10	16.39 2.87	19.94 3.63	23.63 4.44	27.84 5.57	33.17 7.39	38.68 8.55	....	....	....
L-8 (9-in. dowel) 1941	3	¾	4.63 0.98	9.93 2.25	14.58 3.15	18.62 3.77	22.33 4.55	26.52 5.33	31.45 6.20	37.28 7.03	44.77	....	....	....
L-8 (12-in. dowel) 1941	3	¾	4.18 0.95	9.35 2.40	13.82 3.32	17.72 4.00	21.70 4.90	25.95 5.85	30.80 7.00	36.32 8.02	45.55 10.15	51.45 4.65	....	....
L-17	6	¾	3.69 1.06	8.01 2.25	18.56 4.47	33.74 10.06	103.55 68.20	319.12 267.67	584.65 544.00	....	....	....	....	....
L-12	6	¾	9.52 2.18	16.63 3.77	22.23 5.32	27.28 6.83	32.32 8.27	37.35 9.60	42.77 11.63	49.35 14.22	55.10 17.26	65.80 22.10	....	....
L-10	6	¾	8.35 2.25	15.20 3.85	19.75 4.90	24.10 6.05	27.70 6.70	31.70 7.70	36.55 8.85	49.55 14.45	....	....	....	....
L-9	6	¾	20.95 10.15	33.30 17.90	41.28 22.65	48.02 26.10	54.63 28.68	61.28 31.23	68.92 33.90	82.32 43.38	....	....	....	....
L-2	6	¾	7.76 1.84	15.18 3.20	26.78 4.75	37.05 6.08	47.08 7.25	57.35 9.32	69.53 12.70	86.93 22.65	112.28 43.02	151.25 80.58	199.88 125.83	....
L-3	6	¾	5.34 1.37	11.23 2.57	16.21 3.42	21.25 4.35	26.13 5.21	32.52 6.77	43.01 11.27	60.63 19.37	....	....	....	....
L-1	24	¾	9.38 2.17	15.56 3.81	20.35 5.16	24.89 6.42	29.20 7.64	33.83 8.93	39.03 10.51	45.68 12.81	56.56 17.71	....	....	....

NOTE: Values in upper line of each section of table are deflections under load; those in lower line are permanent sets after removal of load.

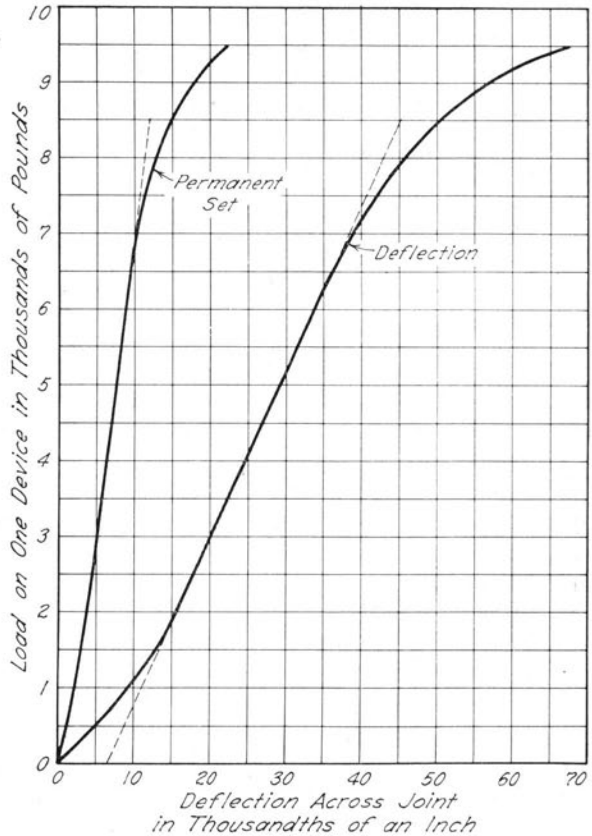


FIG. 61. AVERAGE LOAD DEFLECTION AND PERMANENT SET CURVES FOR L-1 LOAD TRANSMISSION DEVICE (ILLINOIS DIVISION OF HIGHWAYS TESTS)

many data being available for that device because it was used as a standard of comparison in tests of many of the devices submitted. Figure 61 is a graph showing composite load deflection and load permanent set curves obtained by averaging the results from all tests made on the wing anchor device.

Referring to Fig. 61, it will be seen that both the deflection curve and the permanent set curve have the same general shape, the former having a flatter slope and its changes in curvature being more pronounced. Beginning at the origin, the load deflection ratio is relatively low, gradually increasing up to a load of 2,000 lb., from whence it remains constant up to a load of approximately 6,500 lb., from which point the load deflection ratio, or the slope of the curve, uniformly decreases up to approximately 9,500 lb., the average load at which the concrete test specimens failed.

During the lower end of the curve, the component parts of the device adjusted themselves and any play or lost motion was taken up. This is particularly characteristic of devices such as the wing anchor, which consist of a bar fitted into a metal socket. Devices such as the conventional dowel bar, when the bar is embedded directly in the concrete, do not always exhibit this lost motion, especially when the concrete is properly compacted around the bar. Where the curve indicates lost motion and a considerable portion of the curve lies in a straight line, as is the case in Fig. 61, the effective amount of lost motion can be determined by extending the straight line part of the curve to the abscissa and reading the deflection at the point of intersection. The lost motion indicated in Fig. 61 is 0.0065 in.

For a considerable range in load, as indicated by the constant slope portion of the curve, elastic deformations occur, and it may be assumed that, so long as the loads which a device is required to carry in service are below the upper limit of this range, the device will not be damaged, provided of course that constant repetition of these loads does not introduce other factors. The upper limit of this range, or the point at which the deflection is no longer proportional to the load, may be called the elastic limit, as it is the point at which definite elastic failure begins. The elastic limit for the L-1 device, as shown in Fig. 61, is approximately 6,500 lb. It is interesting to note that both the load deflection curve and the load permanent set curve are in close agreement in this respect. Obviously, the elastic limit must be higher than the load which the device may be required to transmit, in order for the device to be satisfactory, and the greater the difference between the two the greater the margin of safety of the device.

All devices do not yield load deflection curves such as those shown in Fig. 61. Some, particularly the conventional dowel bar, give curves which have a gradual curvature beginning at the origin, probably due to a slight progressive failure of the concrete under the bar which has the effect of increasing the length of bar subject to bending. In those cases, the elastic limit is better indicated by the load permanent set curves, which are much less influenced by secondary failures than the load deflection curves.

The slope of the straight line portion of the curve is of great importance, because it is a measure of the stiffness of the device. The stiffer a device, the more effective it is in transferring load, a fact that will be observed from an analysis of Table 17. It is desirable, in connection with analytical relations to be discussed later, to express the stiffness as the deflection per pound of load, or the reciprocal of the slope of the load deflection curve. Referring to Fig. 61, the intersection of the straight line portion of the curve with the abscissa is 0.0065 in., and the deflection at a load of 6,000 lb. is 0.0338 in. The ratio of deflection to load then is $(0.0338 - 0.0065) \div 6{,}000 = 0.00000455$ in. per lb. Since some devices do not yield load deflection curves having a constant slope, it is necessary in those cases to evaluate the effect of slope by other means. This is covered fully in the discussion of the method for estimating the amount of load transfer by successive approximations, given later (pages 109-10).

The ultimate load at which the concrete fails is a relative measure of the

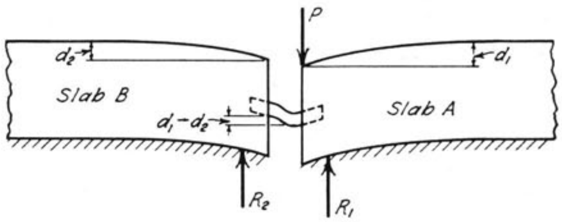


FIG. 62. SKETCH SHOWING CONDITION (EXAGGERATED) OF LOAD TRANSFER THROUGH DOWEL UNDER THE LOAD P AND THE RESULTANT SUBGRADE REACTIONS R_1 AND R_2, THE LATTER OF WHICH ALSO IS THE AMOUNT OF LOAD CARRIED THROUGH THE DOWEL

ability of the device to distribute to the concrete the reactions due to the load transferred. In this respect also it is essential that a device have a good margin of safety. The average load at concrete failure for the specimens containing wing anchor devices, as shown by Fig. 61, is approximately 9,500 lb.

Having determined these characteristics for any load transmission device, that device may be compared with a standard device whose performance has been established by successful use. They also may be used to determine, at least relatively, the performance a device may be expected to give in service, by substituting their numerical values in certain analytical relations developed by the Division of Highways. The derivation of these relations follows.[7]

In Fig. 62 are shown two slabs, A and B, resting on an elastic subgrade, separated by a joint and mutually supported by a load transmission device. Let

P = Load on Slab A near the joint and directly above the load transmission device.

R_1 = Resultant subgrade reaction under Slab A, or part of load not transferred through load transmission device.

R_2 = Resultant subgrade reaction under Slab B, or part of load transferred through load transmission device.

d = Deflection of Slab A directly under Load P, which would occur if the load transmission device were absent.

d_1 = Deflection of Slab A under Load P with load transmission device operative.

d_2 = Deflection of Slab B at load transmission device corresponding to d_1 in Slab A.

s = Deflection of load transmission device in in. per lb. of load transmitted, as determined from the straight line portion of the load deflection curve.

L = Lost motion in in., as determined from load deflection curve.

[7] The reader's attention is called to the fact that this method of analyzing test results from load transmission tests was developed by the Illinois Division of Highways, independently of the method described in Section 9(d) (page 62), which was used by the University of Illinois to analyze their test results. This explains the difference in notation and terminology used in the two discussions.

Since for an elastic subgrade, the subgrade reaction is directly proportional to the slab deflections, the following relations can be written:

$$\frac{d}{P} = \frac{d_1}{R_1} = \frac{d_2}{R_2} \tag{5}$$

$$R_1 = \frac{Pd_1}{d} \tag{6}$$

$$R_2 = \frac{Pd_2}{d} \tag{7}$$

$$d_1 + d_2 = d \tag{8}$$

$$R_1 + R_2 = P\,. \tag{9}$$

The difference $(d_1 - d_2)$ between the deflections of the two slabs represents the amount of bending or deflection of the load transmission device in transmitting a load R_2. From the load deflection curve this deflection is $sR_2 + L$. Equating these two expressions,

$$sR_2 + L = d_1 - d_2\,. \tag{10}$$

Substituting in (10) the value of d_1 from (8)

$$d_2 = \frac{d - sR_2 - L}{2}\,.$$

Substituting this value for d_2 in (7) and simplifying

$$R_2 = \frac{Pd_2}{d} = \frac{P\,(d - sR_2 - L)}{2d}$$

$$R_2 = \frac{P\,(d - L)}{2d + sP}\,. \tag{11}$$

To find the probable amount of load a device may be called on to transmit under given conditions, it is only necessary to substitute in (11) the proper numerical values for the wheel load, P, the deflection of a free slab, d, the lost motion, L, and the deflection per lb. load, s, determined from the load deflection curve.

For the purpose of comparing various devices, P is assumed to be 8,000 lb.,[8] the maximum legal wheel load permitted by Illinois statutes. If it is

[8] This was the maximum legal wheel load in Illinois at the time this analysis was made. The law was amended in 1945 to permit an 18,000-lb. axle load, which is the practical equivalent of a 9,000-lb. wheel load.

desired to take into account dynamic effects of a moving wheel load, this may be done by multiplying the static wheel load by the proper factor. It has been found that relative values are changed but little by introducing corrections for dynamic loads, so, for simplicity, the static load is used here.

The deflection of a free slab, d, is computed from the formula developed by Dr. H. M. Westergaard for the deflection of an unbroken edge under a concentrated load,

$$d = \frac{0.433\,P}{kl^2} \tag{12}$$

where

d = deflection in in. at point of application of load.
P = concentrated load applied to edge of slab.
k = modulus of subgrade reaction.
l = radius of relative stiffness of slab.

In this work d is computed for a 7-in. edge, since this is the thickness of the edges formed by a joint in the standard 9-9-7-9-9 pavement widely used in Illinois. A value of 50 lb. per cu. in., which corresponds to a relatively weak subgrade, is chosen for k, because a load transmission device is required to transfer more load under that condition than when the subgrade is more resistant. The deflection for a uniform 7-in. slab on a relatively weak subgrade and an 8,000-lb. wheel load is 0.0523 in., as computed from Equation (12).

The use of Equation (11) is illustrated by the following computations for the L-1 device. The values for the various known quantities in the equation are

$$\begin{aligned} P &= 8{,}000 \text{ lb.} \\ d &= 0.0523 \text{ in.} \\ L &= 0.0065 \text{ in.} \\ s &= 0.00000455 \text{ in. per lb.} \end{aligned}$$

Substituting these values in (11),

$$R_2 = \frac{8{,}000\,(0.0523 - 0.0065)}{0.1046 + 0.00000455 \times 8{,}000} = 2{,}596 \text{ lb.}$$

In other words, the L-1 device, under the conditions assumed, would be required to transfer 2,596 lb. of an 8,000-lb. static wheel load, or 32.4 per cent.

The load transfer for a device also may be determined by the method of approximation. With this method, the load transfer is computed for an arbitrarily assumed deflection across the joint, and the deflection from the load deflection curve corresponding to this amount of load transfer is compared with the assumed deflection. This procedure is repeated until a load transfer is found whose corresponding deflection from the curve is the same as the assumed deflection. The method is applicable to any load deflection curve, but is generally used where the curve has no straight line portion which will permit the constants for Equation (11) to be determined. However, for the purpose of illustrating the method, it will be used to check the value for load transfer computed by Equation (11) for the L-1 device.

For the first approximation, assume a deflection of 0.022 in. Then

$$\begin{aligned} d_1 - d_2 &= 0.022 \\ d_1 + d_2 &= 0.0523 \\ 2d_1 &= 0.0743 \\ d_1 &= 0.03715 \\ d_2 &= 0.01515 \\ R_2 = \frac{Pd_2}{d} &= \frac{8{,}000 \times 0.01515}{0.0523} = 2{,}317 \text{ lb.} \end{aligned}$$

Referring to the load deflection curve in Fig. 61, the deflection corresponding to this load is 0.017 in., which does not agree with the assumed deflection, 0.022 in.

As a second approximation, assume a value of 0.020 in. Then,

$$\begin{aligned} d_1 - d_2 &= 0.020 \\ d_1 + d_2 &= 0.0523 \\ 2d_1 &= 0.0723 \\ d_1 &= 0.03615 \\ d_2 &= 0.01615 \\ R_2 &= \frac{8{,}000 \times 0.01615}{0.0523} = 2{,}470 \text{ lb.} \end{aligned}$$

and the corresponding deflection from Fig. 61 is 0.0178 in., which is not a sufficiently close check.

As a third approximation, assume a deflection of 0.0184 in. Then

$$\begin{aligned} d_1 - d_2 &= 0.0184 \\ d_1 + d_2 &= 0.0523 \\ 2d_1 &= 0.0707 \\ d_1 &= 0.03535 \\ d_2 &= 0.1695 \\ R_2 &= \frac{8{,}000 \times 0.01695}{0.0523} = 2{,}593 \text{ lb.} \end{aligned}$$

which corresponds to the assumed deflection as closely as the curve can be read. It is interesting to note that this value is within 3 lb. of that computed from Equation (11).

Having determined the elastic limit of a device and computed the probable load transfer, one may compute the safety factor by dividing the elastic limit by the load transfer. For the L-1 device this is 6,500 divided by 2,596, which equals 2.50. The safety factor with respect to concrete failure is 9,500 divided by 2,596, or 3.66. The percentage of an 8,000-lb. static wheel load which the L-1 device may be required to transfer under assumed conditions has already been shown to be 32.4.

All these characteristics, important in deciding whether a device is satisfactory, should be considered collectively. For instance, one device may show a higher load transfer than another, but when the elastic limits are also considered, the second device may be superior. This is true of the conventional dowel and the L-1 device. The former has a load transfer of 39.1 per cent, or 3,128 lb. of an 8,000-lb. static wheel load, and an elastic limit of 3,000 lb. The latter has a load transfer of 32.4 per cent, or 2,596 lb. of an 8,000-lb. static wheel load, and an elastic limit of 6,500 lb. It is readily seen from these figures that, comparatively at least, the conventional dowel bar is in danger of elastic failure when transmitting the computed load, while the wing anchor device has a considerable margin of safety.

The percentage of load transfer is also an indication of the efficiency of a device in transferring load. If a device is 100 per cent efficient, it will transfer 50 per cent of the applied load, assuming that only one unit is active. Hence, if the computed percentage of load transfer is divided by 50 per cent, it will give the efficiency of the device in terms of the theoretical maximum. For example, the efficiency for the L-1 device is $\frac{32.4}{50} = 0.65$. This ratio is analogous to "relative effectiveness" used in the discussion of the results of the University of Illinois tests.

The average characteristics of the load transmission devices tested by the Division of Highways, obtained by analyzing the average data from all tests, are given in Table 17. It is important to note that the devices shown in the upper part of the table were tested with the end sections fully supported by the steel plates, while in the tests of the others the supporting plates were set back 4 in. from the joints. There are no data to establish the effect of the position of the supporting plates, but it is believed that fully supported end sections will give higher values for load transfer and elastic limit. The joint width used in the tests was ¾ in., except for the L-5 and L-6 devices. The L-5 device was tested with a 1-in. joint; the L-6 device with both ½- and 1-in. joints. When no values were given for L and s, these characteristics could not be determined from the test data. The importance of joint width, or the span through which a device operates, is clearly shown by the results from tests on the conventional dowel across ½-in., ¾-in., and 1-in. joints, respectively. Both the percentage of load transfer and the elastic limit vary inversely with the joint width.

TABLE 17

CHARACTERISTICS OF LOAD TRANSMISSION DEVICES

(Illinois Division of Highways Tests)

Device	Number of Speci-mens Tested	Width of Joint in.	Width of Specimen in.	L in.	S in. per lb. x 10^8	Yield lb.	Concrete Strength lb. per sq. in.	Load at Failure lb.	Load Transfer per cent	Net Efficiency per cent
DEVICES LISTED IN TABLE 15										
L-6[1]	6	½	10	......	...	6,300	3,749	6,200	40.6	.81
L-6[2]	6	½	10	......	...	5,500	3,655	5,300	36.3	.73
L-6[3]	6	½	10	......	...	3,000	3,655	4,300	24.7	.49
L-5	6	1	10	......	...	5,000	3,320	10,400	37.8	.76
L-4	6	¾	12	......	...	9,250	5,484	10,500	39.0	.78
L-9	6	¾	10	0.012	...	5,000	4,653	8,125	28.1	.56
L-6[4]	6	½	10	......	...	3,200	3,718	4,250	29.7	.59
L-6[5]	6	1	10	......	...	3,000	3,865	4,000	21.6	.43
Conventional dowel[6]	6	½	10	......	...	3,500	3,725	9,280	44.0	.88
Conventional dowel[6]	12	¾	10	......	...	3,000	4,818	8,764	39.1	.78
Conventional dowel[6]	6	1	10	......	...	2,250	3,560	7,700	37.4	.75
DEVICES LISTED IN TABLE 16										
L-11	6	¾	12	0.0086	401	7,200	5,619	12,300	32.0	.64
L-7	6	¾	12	0.0060	375	6,000	5,690	10,300	34.4	.69
L-8 (9-in. dowel) 1940	7	¾	12	0.0019	364	6,600	5,579	8,700	37.7	.75
L-8 (9-in. dowel) 1941	3	¾	12	0.0017	...	6,000	5,381	9,300	36.4	.73
L-8 (12-in. dowel) 1941	3	¾	12	0.0017	...	6,000	5,381	9,600	36.9	.74
L-17	6	¾	12	......	...	2,200	5,735	.....	28.6	.57
L-12	6	¾	12	0.0081	476	6,500	4,860	9,300	31.0	.62
L-10	6	¾	12	0.0042	726	6,200	5,619	10,100	29.6	.59
L-9	6	¾	12	0.0215	665	6,300	5,593	8,000	21.5	.43
L-2	6	¾	12	0.0062	1,016	5,000	5,886	10,700	24.8	.50
L-3	6	¾	12	0.0012	508	5,400	5,886	7,500	35.2	.70
L-1	24	¾	12	0.0065	455	6,443	5,588	9,486	32.4	.65

[1] Modified plate dowel joint. Plate dowel was integral part of side walls of joint.
[2] Plate dowel joint with ¼-in. x 1½-in. plate. Tests made at normal joint width.
[3] Same specimens as (2). Tests made with joint opened 0.22 in. beyond normal to determine action when slabs are pulled apart by contraction of concrete.
[4] Plate dowel joint with 3/16-in. x 2½-in. plate.
[5] Plate dowel joint with ¼-in. x 3-in. plate.
[6] ¾-in. round hot rolled bar, 24 in. long.

(d) Opening-Closing Tests of Copper Seals

Because, as stated heretofore, the test procedure was not exactly the same and observations were not made at the same cycles for all specimens, it is not possible to present the test results in the same manner as for the University tests in which a standard test procedure was followed. Test results given in Tables 18 to 23, inclusive, show the distance each specimen was opened and closed; the number of cycles at which first failure and total failure occurred in the soldered junction between the top seal and the end seals, if the joint possessed this feature; the number of cycles at which the first crack occurred in the copper; the total number of cycles to which the joint was subjected; and a description of the final condition of the copper seal.

(1) *J-1* Expansion Joint. The data for this joint are given in Table 18. It will be noted that the initial failures of the soldered junction between top and end seals occurred much earlier on Specimens 1, 2, and 3 than on the remainder of the specimens. The junction on the joints in Specimens 1, 2, and 3 was made by mitering the top and end seals, butting them together and soldering. This made a weak junction because the solder had to take all the strain. In the other specimens, the top seal was lapped and soldered over the end seal. That this construction strengthened the corner is shown definitely by Table 18; failures in the lapped junctions occurred only after a large number of cycles of opening and closing, while the butted junctions all failed during the first few cycles. Strengthening the junction between the top and end seals, however, resulted in the strains around the corner being transferred to the copper in that vicinity. Cracks in the copper developed earlier in the joints with the lapped corners, and the first cracks invariably occurred immediately adjacent to the laps. Any failure in the vicinity of the junction, whether in the solder or copper, had the effect of relieving the strain at that point. Only Specimen 7, of the seven specimens with lapped corners, showed total failure of the soldered junction and that occurred at 3,000 cycles. In the others of this type, the cracks in the copper adjacent to the soldered laps relieved the strain so that the solder did not fail. On the other hand, in two out of three of the specimens with mitered and soldered junctions, total failure of the soldered joint occurred well in advance of the first crack in the copper.

In all but one case, the first crack in the copper occurred at a much greater number of cycles than the minimum required by the specifications. While subsequent developments, which are discussed in Chapter IV (page 139), showed conclusively that this test was not nearly so severe as the forces which act on a joint in a pavement, since it was found from field surveys that none of the seals withstood those forces, at the time of the tests this seal was judged to be satisfactory. Test results show that this seal was superior to others tested, as may be seen from a study of Tables 18 to 23, inclusive.

(2) *J-2* Air Chamber Joint. Data for this joint are given in Table 19. The measured expansion space was ⅞ in. Two of the joints were closed and

TABLE 18

RESULTS OF OPENING-CLOSING TESTS ON J-1 EXPANSION JOINT

(Illinois Division of Highways Tests)

Specimen	Movement in.	Soldered Junction: Construction	Soldered Junction: First failure cycles	Soldered Junction: Total failure cycles	First Copper Failure cycles	Total Cycles Run	Final Condition
1	5/8	Mitered	1	389	1,200	2,800	Total length of cracks, 6 in. (around soldered junction).
2	5/8	Mitered	3	113	740	3,000	Total cracks 1½ in.; longest 1 in.
3	29/32	Mitered	1	—	400	3,000	Total cracks 1 in.
4	5/8	Lapped	214	—	52	2,800	Four cracks totaling 3 in. around soldered junction.
5	¾	Lapped	43	—	137	3,000	Total cracks ¾ in.
6	13/16	Lapped	45	—	30	3,000	Badly damaged.
7	13/16	Lapped	100	3,000	580	3,000	Soldered junctions failed completely. Total cracks in copper 2 in.
8	15/16	Lapped	28	—	990	3,035	Soldered junctions badly damaged. Total cracks in copper 3 in.
9	11/16	Lapped	70	—	200	3,000	Five cracks in copper totaling 3½ in.
10	11/16	Lapped	..	..	100	3,000	Ten cracks in copper totaling 5⅛ in.

TABLE 19

RESULTS OF OPENING-CLOSING TESTS ON J-2 EXPANSION JOINT

(Illinois Division of Highways Tests)

Specimen	Length of Joint ft.	Movement in.	First Copper Failure cycles	Total Cycles	Final Condition
1	2	¾	18	375	Complete failure of seal; total length of cracks 23 in.
2	1	15/16	26	162	Complete failure of seal; total length of cracks 12 in.
3	1	15/16	6	143	Complete failure of seal; total length of cracks 12 in.
4	3	¾	8	135	Seventeen cracks in copper totaling 36¾ in.; total failure.

TABLE 20

RESULTS OF OPENING-CLOSING TESTS ON BITUMINOUS-FILLED, COPPER-SEALED J-2 EXPANSION JOINT

(Illinois Division of Highways Tests)

Specimen	Movement in.	First Copper Failure cycles	Total Cycles	Final Condition
1	⅜	15	40	Complete failure.
2	⅜	41	78	Complete failure.
3	5/16	32	79	Complete failure.
4	¼	96	172	Complete failure.

opened ¾ in., and for the other two the closure was 15⁄16 in. The joints in Specimens 1, 2, and 3 had a plate and sleeve type end closure, with a copper clip to seal the crack between it and the top seal. This clip was not satisfactory, because it was bent out of shape during the first closing and subsequently did not cover the opening. The localized strains near the ends of the top seal, which were produced in other joints by soldering copper end seals to the top seal and were the cause of the initial failures, were not present in this joint. Nevertheless, initial failures occurred at from six to 26 cycles. The joint in Specimen 4 had copper seals butt jointed and soldered to the top seal. Complete failure of both soldered corners occurred during the second cycle, and the initial failure in the copper occurred at eight cycles. The tests were continued until the seal had failed completely. Complete failure of the specimens occurred at 135, 143, 162, and 375 cycles.

(3) Bituminous Filled *J-2* Expansion Joint. The seal on this joint was enclosed in a block of bituminous premolded mastic which permitted only limited movement. The specimens were first compressed from ¼ to ⅜ in., the joints reopened, the mastic above the seal removed and the seal cleaned with a solvent so that it could be observed during the opening-closing test. The top and end seals were fitted by mitering but the junction between them was not soldered, the mastic being expected to seal the opening at that point. The results of tests on four specimens are given in Table 20. Initial failures in the copper occurred at from 15 to 96 cycles. Complete failure of the seal developed very early; in one specimen at 40 cycles, one at 78 cycles, one at 79 cycles, and the other at 172 cycles. The early failures were attributed to the distortion of the seal caused by the action of the bituminous material beneath it when the joint was closed. The action undoubtedly produced severe strains in the copper.

(4) *J-4* Expansion Joint. The test results for six specimens containing this point are given in Table 21. The junction between the top and end seals on this joint was a soldered lap, and this construction produced severe strains in the vicinity of the junction. In every specimen, the soldered junction failed completely during the first cycle. The trough of the seal, which was made in the form of the letter M, was shallow, and after the first closure it remained closed so that all the movement was taken up by one or the other of the folds at the top of the seal. This concentration of movement at one point caused failures in the copper to develop early. One specimen had its first failure at 16 cycles, and the other five specimens failed initially at 10 cycles or less. Complete failure of the seals occurred at from 100 to 150 cycles.

(5) *J-6* Expansion Joint. The results of tests of four specimens containing this joint are given in Table 22. The copper seal on this joint was continuous over the top and ends, the seal being drawn on a 3-in. radius at the corners and extending down the ends to the bottom of the joint. However, only Specimens 3 and 4 contained joints complete with end seals, the joints

TABLE 21
RESULTS OF OPENING-CLOSING TESTS ON J-4 EXPANSION JOINT
(Illinois Division of Highways Tests)

Specimen	Movement in.	Soldered Junction		First Copper Failure cycles	Total Cycles	Final Condition
		First failure cycles	Total failure cycles			
1	21/32	1	1	6	103	Complete failure of copper seals.
2	21/32	1	1	1	100	Complete failure of copper seals.
3	5/8	1	1	10	112	Complete failure of copper seals.
4	21/32	1	1	10	110	Complete failure of copper seals.
5	5/8	1	1	2	130	Complete failure of copper seals.
6	5/8	1	1	16	150	Complete failure of copper seals.

in Specimens 1 and 2 having only a top seal and being open at the ends. The effect of the end seals in introducing additional strains in the copper is perfectly obvious from the test results. Specimens 1 and 2, without end seals, showed initial failures at 80 and 38 cycles, respectively, while initial failures occurred in Specimens 3 and 4, with end seals, at five and three cycles, respectively. All the specimens were run to complete failure of the seal, which occurred at from 157 to 328 cycles, three of the specimens failing at less than 200 cycles. The results indicate that the end seals undoubtedly influence complete failure much less than initial failure, because once failures occur in the vicinity of the junction between top and end seals the additional strain is relieved, and from there on the joint acts like one without end seals.

TABLE 22
RESULTS OF OPENING-CLOSING TESTS ON J-6 EXPANSION JOINT
(Illinois Division of Highways Tests)

Specimen	Movement in.	First Copper Failure cycles	Total Cycles	Final Condition
1[1]	5/16	80	328	Complete failure of seal.
2[1]	3/8	38	174	Complete failure of seal.
3[2]	11/32	5	157	Complete failure of seal.
4[2]	3/8	3	186	Complete failure of seal.

[1] Specimens 1 and 2 contained joints without end seals; joints 12 in. long.
[2] Specimens 3 and 4 contained joints with top and end seals drawn from one piece of copper; joints 18½ in. long.

(6) *J-8* Expansion Joint. Four specimens containing this joint were tested; results are given in Table 23. It was desired to make the tests with the bituminous joint material in place, in order to determine its effect on the copper seal. The shaper did not have sufficient power for the test and the special hydraulic machine had not been built at the time, so a method was devised whereby the test could be made in a universal testing machine. The method was very slow and laborious; only about 20 cycles could be run in one day. For that reason, the tests were not carried to complete failure of the seal, but were stopped after major failures developed. Specimens 1, 2, and 3 were tested in this manner. In testing Specimen 4, the bituminous filler was removed from the joint after initial compression in the testing machine, the specimen placed in the shaper, and the test continued until the copper seal failed completely.

Table 23 shows that initial failure occurred in Specimens 1, 2, and 3 at 36, 18, and 20 cycles, respectively, and at 42 cycles in the case of Specimen 4. Specimens 1 and 3 were run 150 cycles and the test on Specimen 2 was stopped at 100 cycles. All these specimens contained major failures in the seals at that time. Table 23 shows that Specimen 4 failed completely at 1,201 cycles, but major failures had developed much earlier. For instance, at 132 cycles four failures aggregating 6 in. in length had developed, or, in other words, the seal was split for one-third its length.

Table 23
Results of Opening-Closing Tests on J-8 Expansion Joint
(Illinois Division of Highways Tests)

Specimen	Movement in.	First Copper Failure cycles	Total Cycles	Final Condition[1]
1	3/8	36	150	Five cracks in copper seal; major failures.
2	3/8	18	100	Two major failures.
3	3/8	20	150	Three major failures.
4	5/16	42	1,201	Complete failure of copper seal.

[1] On all specimens, the soldered junctions between top seal and end seals failed completely during first cycle.

(e) Anchorage Test of Copper Seals

The results of the anchorage tests are given in Table 24. The J-1 seals in four specimens failed in tension along a line through the perforations in the flange where the section is reduced. The average maximum load to produce failure was 2,128 lb. per ft. of joint. Loads for individual specimens varied from 2,030 to 2,270 lb. per ft.

Failure of the specimens containing J-2 seals occurred at an average maximum load of 1,660 lb. per ft. Individual specimens failed at

TABLE 24
RESULTS OF ANCHORAGE TESTS ON EXPANSION JOINT SEALS
(Illinois Division of Highways Tests)

Specimen	Load, lb. per ft.			Remarks
	At initial slip	Maximum	Average maximum	
J-1 EXPANSION JOINT (FLANGE EMBEDMENT 1¼ IN.)				
1	Not recorded	2,030		All seals failed in tension at point of minimum section along line of perforations in flange.
2	Not recorded	2,120		
3	Not recorded	2,090		
4	Not recorded	2,270	2,128	
J-6 EXPANSION JOINT (FLANGE EMBEDMENT 1⅙ IN.)				
1	2,230	3,190		Copper failed by tearing.
2	2,000	2,760		Copper failed by tearing.
3	1,800	1,950	2,633	Failed by flange lugs pulling out of concrete.
J-2 AIR-CHAMBER JOINT (FLANGE EMBEDMENT ¾ IN.)				
1	Not recorded	1,380		In all specimens the flange of the seal pulled out of the concrete.
2	Not recorded	2,350		
3	Not recorded	1,567		
4	Not recorded	1,393	1,660	
J-4 EXPANSION JOINT (FLANGE EMBEDMENT ⅞ in.)				
1	Not recorded	1,890		In all specimens the concrete cracked and the seal pulled out.
2	Not recorded	1,980		
3	Not recorded	2,050		
4	Not recorded	2,090		
5	Not recorded	2,190		
6	Not recorded	2,275		
7	Not recorded	2,050		
8	Not recorded	2,260	2,099	

loads of 1,380 to 2,350 lb. per ft., the copper seal pulling out of the concrete in every case. Failure in bond was preceded by a tearing of the small triangular projections on the flange of the seal, whose purpose was to develop mechanical bond. It may be concluded, then, that initial failure was by tearing of the copper. Had the design of the flange been properly balanced, the test results, without question, would have been higher. In this connection, it should be noted that the flange on the J-2 joint was narrow, not providing the same depth of embedment as the seals on some of the other joints.

Failure of the eight J-4 specimens occurred at loads ranging from 1,890 to 2,275 lb. per ft. of joint, the average load being 2,099 lb. per ft. In every case, failure occurred by cracking of the concrete beneath the seal, allowing the flange to pull out.

In testing the specimens containing J-6 joint seals, it was noted that initial failure was of the nature of a slip between the copper and the concrete, which occurred at loads somewhat below those which produced ultimate failure. The load at initial slip varied from 1,800 to 2,230 lb. per ft. of joint. After initial failure, the load became eccentric, the final failure being both in bond and tension, which resulted in a pulling out and tearing of the lugs which formed a part of the flange. Ultimate failure occurred at loads from 1,950 to 3,190 lb. per ft., or an average of 2,633 lb.

While it is not to be expected that the seals on expansion joints will be straightened out and placed under tension as in the tests, nevertheless the test results are valuable in that they show, in a relative way, how well a seal may be expected to resist tensile forces induced by the pressure of heavily loaded wheels acting through accumulations of foreign material packed around the seal. It appears from the test results that, if the concrete is properly compacted around the seals, no trouble in this respect should be experienced from any of the joints. It is pointed out, however, that honeycomb and slumping of the concrete under the seal will have a proportionately greater effect with short flanges than with flanges which provide deeper embedment.

(f) Installation Test

The first metal joints installed in Illinois in 1932 and 1933 had no internal support to augment the stiffness of the side walls, it being thought that the metal walls were rigid enough to resist the pressures produced by placing, compacting, and finishing the concrete adjacent to the joint. Subsequent field investigations proved this assumption to be untrue, as many joints were found in which the available space for expansion had been seriously reduced by collapse of the side walls. The manufacturers furnishing joints were required to correct this defect, and in every case it was done by using some kind of separator, usually a channel-shaped member placed midway between the top and bottom of the joint and extending the full length of the joint.

The installation test, described in sub-section 9(g) (page 85), was devised to determine how well a joint would resist collapse under conditions similar to those encountered in actual practice. The results of tests made on the J-1 and J-4 expansion joints are given in Tables 25 and 26, respectively, and on the J-6 expansion joint in Tables 27 and 28. The values in the tables are the measured joint widths, in 32nds of an inch, at various points after the joint was placed in concrete. Since the measurements naturally included any variations from

TABLE 25
DISTANCE BETWEEN SIDE WALLS OF J-1 EXPANSION JOINT AFTER INSTALLATION TEST
(Illinois Division of Highways Tests)

Distance from Bottom	Distance from East End of Joint, in.														
in.	2	6	10	14	18	22	26	30	34	38	42	46	50	54	58
6	28	28	28	28	28	27	27	27	27	27	26	26	26	26	26
5	25	25	25	24	24	26	24	23	23	25	24	24	24	23	23
4	26	26	26	25	25	25	25	23	24	25	24	24	24	24	24
3	25	25	25	24	24	25	25	24	25	25	25	25	25	25	24
2	24	24	24	23	23	23	23	24	24	24	25	24	25	25	25
1	24	24	24	23	25	23	23	23	24	24	24	24	25	25	25

NOTE: Values are in 32nds of an in. Nominal width 24.

TABLE 26
DISTANCE BETWEEN SIDE WALLS OF J-4 EXPANSION JOINT AFTER INSTALLATION TEST
(Illinois Division of Highways Tests)

Distance from Bottom	Distance from East End of Joint, in.												
in.	3	7½	12	16½	21	25½	30	34½	39	43½	48	52½	57
5	(1)	26	26	(1)	26	26	(1)	27	27	(1)	27	27	(1)
4	24	26	28	26	27	26	(1)	26	26	24	26	26	(1)
3	25	27	27	24	26	26	25	26	26	26	26	26	(1)
2	26	26	26	25	26	26	26	26	26	26	26	25	(1)
1	25	24	24	24	25	26	25	25	25	25	25	26	26
0	24	24	24	24	24	25	25	25	25	24	25	26	24

NOTE: Values are in 32nds of an in. Nominal width 24.
[1] Broken or honeycombed; reading not taken.

the nominal width which might have existed before the test was made, a value greater than the nominal indicates that the joint probably was wider than the nominal at the time of installation. Hence, variations from maximum to minimum probably indicate fairly closely the maximum collapse.

The J-1 joint, in addition to the support afforded by the closed top and bottom of the stool, was provided with a horizontal channel separator, vertical channel separators in the ends of the stool, and metal ferrules where each load transmission device passed through the joint. The nominal expansion space in this joint was ¾ in. at all points measured, except along the line 6 in. above the bottom, where the nominal space was ⅞ in. Since the actual expansion space shown in Table 25 is given in 32nds of an inch, a value of 24 for any point 5 in. or less from the bottom, or 28 for points 6 in. from the bottom, indicates that no collapse took place. The measurements indicate the maximum amount of collapse was $\frac{3}{32}$ in., and at most points the collapse was $\frac{1}{32}$ in. The minimum available expansion space was $\frac{23}{32}$ in.

Support for the side walls of the J-4 expansion joint was afforded by the closed bottom of the stool, dowel ferrules, and a continuous horizontal channel separator. The nominal width of the joint was $\frac{3}{4}$ in., hence in Table 26 a value of 24 indicates that no collapse occurred. It will be noted from Table 26 that most of the readings were greater than 24 and none less than 24. The maximum collapse, as indicated by the range in readings, was $\frac{1}{8}$ in., but in most cases the maximum collapse was $\frac{1}{16}$ in. and the average collapse was $\frac{1}{32}$ in. After the test, the minimum available expansion space was $\frac{3}{4}$ in., indicating that the original width of the joint had been greater than the nominal.

Tests were run at different times on two samples of J-6 expansion joint. Both samples received principal support from dome-shaped indentations in the side walls which, when placed together, held the side walls apart. The first sample submitted had one row of separators 4 in. from the bottom, spaced about 2 ft. apart, and a row $1\frac{1}{8}$ in. from the bottom, spaced alternately 11 in. and 24 in. apart. This sample failed badly in the installation test. As shown by Table 27, the expansion space at one point was only $\frac{1}{16}$ in., as compared with a nominal width of $\frac{1}{2}$ in. At one-third of the points where measurements were made, the expansion space was reduced to half or less of the nominal.

The second sample had a row of separators 4 in. above the bottom, spaced 12 in. apart, and a second row $1\frac{1}{2}$ in. above the bottom, spaced 6 in. apart. It also received additional support at the bottom from a plate spot-welded to the two side walls. Table 28 shows that the additional strengthening improved the joint materially. The minimum available expansion space after test was $\frac{5}{16}$ in., as compared with a nominal space of $\frac{1}{2}$ in. The maximum collapse as indicated by the range in readings was $\frac{3}{8}$ in., but the average was about $\frac{3}{16}$ in. Although an improvement over the first design, the second sample could not be considered satisfactory, when almost 40 per cent of its available expansion space was destroyed during installation. In both samples a considerable amount of mortar had seeped into the joint and deposited on the bottom, streaks on the side walls indicating that it had entered under the copper seal, which was merely set between the side walls and held in place by friction.

TABLE 27

DISTANCE BETWEEN SIDE WALLS OF J-6 EXPANSION JOINT AFTER INSTALLATION TEST

(Illinois Division of Highways Tests)

Distance from Bottom	Distance from East End of Joint, in.																		
in.	1¼	3	7	9¾	12½	16½	20½	23¼	26	30	34	37	39½	43½	47½	50¼	53	57	58¾
6	19	20	19	19	19	21	18	19	19	22	21	20	18	21	18	18	18	18	18
5	20	19	15	13	14	17	14	14	15	20	18	15	15	19	14	13	14	19	19
4	24	20	14	11	11	15	10	9	10	15	12	10	13	16	10	10	12	16	21
3	24	19	13	10	8	14	6	6	7	12	7	7	11	13	6	8	12	17	22
2	20	17	14	9	8	12	7	5	4	7	4	5	13	10	5	7	14	15	19
1	20	17	14	10	8	12	6	4	2	4	3	5	9	5	4	7	11	15	17
0	19	18	14	11	9	8	7	6	4	4	4	5	7	5	5	8	11	15	18

NOTE: Values are in 32nds of an in. Nominal width 16.
Dowels 4 in. from bottom at 3 in., 16½ in., 30 in., 43½ in., and 57 in.
Upper row separator domes 4 in. from bottom at 6 in., 32 in., and 54 in.
Lower row separator domes 1⅛ in. from bottom at 6 in., 19 in., 43 in., and 54 in.

TABLE 28

DISTANCE BETWEEN SIDE WALLS OF J-6 EXPANSION JOINT AFTER INSTALLATION TEST

(Illinois Division of Highways Tests)

Distance from Bottom	Distance from East End of Joint, in.																		
in.	2	6	9	12	15	18	21	24	27	30	33	36	39	42	45	48	51	54	58
5	17	16	18	19	19	19	17	17	16	16	16	16	17	17	17	17	17	16	19
4	17	14	14	14	14	15	14	14	15	15	15	14	14	14	14	13	14	15	22
3	18	13	10	12	11	14	13	13	12	13	13	13	12	16	12	14	11	14	22
2	17	13	10	13	10	14	12	13	12	15	13	15	13	15	12	14	11	13	20
1	17	11	10	13	10	12	11	12	11	13	13	15	11	14	11	13	12	12	22
0	18	11	10	11	10	10	10	10	10	10	10	12	11	12	13	12	12	12	20

NOTE: Values are in 32nds of an in. Nominal width 16.
Dowels 4 in. from bottom at 6 in., 18 in., 30 in., 42 in., and 54 in.
Upper row separator domes 4 in. from bottom at 4 in., 12 in., 24 in., 36 in., 48 in., and 56 in.
Lower row separator domes 1½ in. from bottom at 4 in., 12 in., 18 in., 24 in., 30 in., 36 in., 42 in., 48 in., and 56 in.

(g) Compression Tests

The test results for metal expansion joints are given in Table 29. Since the loading was not continued until concrete failure occurred, the tests do not provide information as to the effect of the joints on the resistance of the concrete slab to axial compression. The University tests indicated that concentrated stresses, probably due to collapsed separators and ferrules not permitting the faces of the joint to come together in uniform contact, were responsible for early failures. The results of Table 29 show that the loads per foot required to close the various joints fully were for the most part fairly uniform and not unreasonably large. Several of the joints closed under relatively light loads, especially Specimen 6 containing a J-2 joint and Specimens 1, 2, 3, and 4 containing the J-6 joint. These joints did not possess sufficient resistance to lateral pressures. The J-2 joint in Specimen 6 was an experimental sample submitted for test. It had no intermediate separators and so obviously lacked stiffness that an installation test was not made. Specimens 1, 2, 3, and 4 contained sections of the first J-6 joint submitted, which collapsed badly under the installation test.

The premolded bituminous cap was left in place during the tests of Specimen 4, J-1 joint, and Specimen 5, J-6 joint. Loads of more than 26,000 lb., or twice that, applied to any of the specimens tested with the cap removed, were required to close these joints, showing that the resistance added by the cap was quite large. The real seriousness of

TABLE 29
COMPRESSION TESTS ON SPECIMENS CONTAINING METAL EXPANSION JOINTS
(Illinois Division of Highways Tests)

Type of Joint	Specimen	Average Closure in.	Total Load lb.	Type of Joint	Specimen	Average Closure in.	Total Load lb.
J-1	1	17/32	9,000	J-2 Air Chamber	1	25/32	13,890
	2	9/16	11,720		2	23/32	16,905
	3	17/32	15,270		3	23/32	11,070
	4	15/32	26,590		4	23/32	11,815
	5	9/16	11,000		5	23/32	7,740
	6	19/32	10,000		6	11/16	3,375
	7	15/32	10,000				
	8	13/32	10,000	J-4	1	21/32	12,970
					2	21/32	11,000
J-6	1	5/16	1,150		3	21/32	12,000
	2	3/8	4,090		4	21/32	12,000
	3	11/32	2,540		5	21/32	10,330
	4	3/8	5,780		6	21/32	10,110
	5	13/32	26,820[1]		7	21/32	10,000
					8	11/16	10,000
					9	21/32	10,000

NOTE: Joints in J-2 specimens 1 and 2, 24 in. long; all others 12 in. long.
[1] Tested with bituminous cap in place; maximum extrusion of cap 1½ in.

this added resistance is that the pressure is concentrated at the top edge of the slab. This condition, which may occur in service long before a joint becomes fully closed, because of the accumulations of dirt which gather in the top of the joint, may account for at least some of the spalling of concrete found along the joints.

The results of compression tests on expansion joints with premolded fillers are given in Tables 30 and 31. Table 30 gives the results for specimens which were progressively compressed until the concrete

Table 30
Compression Tests of Metal Sealed Premolded Expansion Joints
(Illinois Division of Highways Tests)

Joint	Specimen	Thickness of Filler in.	Load at Half Original Thickness lb.	Concrete Failure Closure in.	Concrete Failure Load lb.	Description of Failure
J-10 (¼-in. x 1½-in. dowel plate)	1	½	16,000	0.353	200,000	Concrete split along plane of plate dowel.
	2	½	16,000	0.365	200,000	Concrete split along plane of plate dowel.
	3	½	20,000	0.351	190,000	Concrete split along plane of plate dowel.
J-10 (Dowel plate part of side walls)	1	½	......	0.245	100,000	Concrete split along plane of plate dowel.
	2	½	50,000	0.292	147,000	Concrete split along plane of plate dowel.
	3	½	50,000	0.266	130,000	Concrete split along plane of plate dowel.
J-8	1	½	32,000	0.323	90,000	Concrete badly split at several points.[1]
	2	½	35,000	0.370	145,000	Concrete badly split at several points.[1]
	3	½	36,000	0.358	165,000	Concrete badly split at several points.[1]
	4	½	36,000	0.379	190,000	Concrete badly split at several points.[1]
J-10 (2¼-in. x 3/16-in. dowel plate)	1	½	16,000	0.347	96,500	Concrete split along plane of plate dowel.
	2	½	20,000	0.349	129,000	Concrete split along plane of plate dowel.
	3	½	16,000	0.344	83,000	Concrete split along plane of plate dowel.
	4	½	20,000	0.290	41,910	Concrete split along plane of plate dowel.
	5	½	16,000	0.367	150,000	Concrete split along plane parallel to plate dowel.
J-10 (3-in. x ¼-in. dowel plate)[2]	6	1	25,000	0.664	80,000	Concrete split along plane of plate dowel.
	7	1	25,000	0.667	96,780	Concrete split along plane of plate dowel.
	8	1	25,000	0.659	98,000	Concrete split along plane of plate dowel.
	9	1	......	0.516	40,000	Concrete split along plane of plate dowel.
	10	1	......	0.461	30,000	Concrete split along plane of plate dowel.
	11	1	......	0.245	6,000	Concrete split along plane of plate dowel.

[1] Location of breaks in J-8 specimens indicate they were influenced by flanges on seals and stiffeners.
[2] Specimens 9, 10 and 11 had purposely deformed expansion sleeves.

failed. The load required to close the joint to half its original thickness was reasonably uniform in most cases, as was to be expected because the joint material was generally of the same type, the load ranging from 16,000 to 25,000 lb. Two joints required considerably higher loads, due to certain features of their construction. The modified J-2 joint was equipped with a sponge rubber filler to seal the expansion space in which the load transmission elements moved during expansion of the concrete. This rubber filler, being confined, required a relatively large load to compress it. The additional resistance offered by the J-8 joint was attributed to the sheet metal stiffeners placed on the top and bottom of the joint and to the hard bituminous premolded asphalt cap placed over the top seal.

Resistance to closure built up rapidly beyond closure to half the original joint thickness, as evidenced by the total closure and load at which failure of the concrete occurred. Total closure was never greater than 75 per cent of the original joint thickness, but loads required to produce this compression ranged from two to 12 times those that produced half closure.

All the joints had features which apparently influenced the failure of the concrete. The J-10 joint specimens failed by splitting of the concrete along the plane of the continuous plate dowel, and the J-8 joint specimens split along the planes of the flanges on the metal stiffeners, all at much lower loads than would be expected from the compressive strength of the concrete. The average compressive strength of the concrete in these specimens was approximately 3,700 lb. per sq. in., which, in a specimen having an area in compression of 84 sq. in., would be expected to give a breaking load of about 311,000 lb. All the specimens failed at loads of 200,000 lb. or less.

It will be noted that those specimens which required more than 25,000 lb. to close the joint to half its original thickness failed at lower loads than the others, and the total closure of the joint was less, indicating that the features mentioned above as adding to the resistance of the joint produced concentrated stresses which caused early failure. An example of the effect of such concentrations of stress may be seen in Table 30, by a comparison of the results from Specimens 6, 7, and 8 with Specimens 9, 10, and 11, all containing 1-in. J-10 joints. The sheet metal sleeve which enclosed the continuous dowel plate on this joint was so light that it was obvious that the sleeve would become seriously deformed under normal shipping and handling operations. In order to determine the effect of such damage, the sleeves on the joints in Specimens 9, 10, and 11 were purposely deformed before placing. Specimen 9, in which the sleeve was only slightly bent, failed at a load less than half the average load required to break Specimens 6, 7, and 8, which contained joints with undeformed sleeves. Specimen 11, in which the sleeve was badly deformed, failed under a load of only 6,000 lb. at a total closure of less than one-fourth the original thickness of the joint. These results, which show the importance of uniform stress distribution, indicate that joints should be designed so that conditions which may lead to stress concentrations will not develop during shipping, handling and placing.

Results of a special test conducted on three specimens containing the J-10 joint, to show the effect of dirt entering the space above the plate dowel, are given in Table 31. In these tests, a load of 50,000 lb. was applied to the specimen and removed, the joint opened 0.22 in., the amount it might be expected to open in service, and the space above the load transmission plate and between the filler and the concrete filled with fine silt. Compressive loads were then applied until the concrete failed, following the same procedure used in testing the specimens listed in Table 30.

The load required to close the joint an amount equal to half the original thickness of the joint filler agrees fairly well with the results shown in Table 30. The closure at 50,000 lb., being only slightly over 0.25 in. in all cases, shows how rapidly the resistance of the material increased after it was compressed to half its original thickness. After silt was put in the joint, failure occurred at closures of less than ¼ in. and at loads of 110,000, 50,000, and 34,000 lb. As shown in Table 30, J-10 joint Specimens 1, 2, and 3, tested without silt, failed at loads of 200,000, 200,000, and 190,000 lb., respectively. This difference indicates the effect of incompressible material in the joint, especially when it enters only the upper half and produces an eccentric loading.

TABLE 31
SPECIAL COMPRESSION TESTS ON J-10 EXPANSION JOINT
(Illinois Division of Highways Tests)

Specimen	Thickness of Filler in.	Compression before Putting Silt in Joint		Concrete Failure after Putting Silt in Open Joint	
		Load at 0.25-in. closure lb.	Closure at 50,000-lb. load in.	Load lb.	Closure in.
1	½	23,000	0.279	110,000	0.245
2	½	28,000	0.280	50,000	0.182
3	½	22,000	0.294	34,000	0.150

NOTE: All specimens failed by splitting along plane of ¼-in. x 1½-in. plate dowel.

(h) Tests of Bituminous Premolded Fiber Joint Filler

The material used as a filler for some of the metal-sealed joints submitted for approval, and alone as a joint after the use of air-chamber joints was discontinued in 1938, is made from cane fibers pressed into a resilient board which is saturated with an asphaltic compound. It is claimed by its manufacturers that this material will not extrude when compressed; that the asphaltic compound effectively protects the fiber board against such destructive agencies as air, moisture, heat, and recurrent cycles of freezing and thawing; that the material adheres permanently to concrete so that it is aided in its recovery when the concrete adjacent to it contracts, and is held firmly in place so that it will not work loose and be displaced. The specifications governing the quality of the material required samples to pass the following tests:

(1) Asphaltic Content. A sample of approximately 45 g. taken from material dried for 4 hr. in an oven at 125 to 135 deg. F., when tested in a Soxhlet extraction apparatus with carbon tetrachloride as the solvent, the residue being thoroughly dried at a temperature of 125 to 135 deg. F., shall show not less than 35 per cent bitumen by weight.

(2) Freezing and Thawing Test. A specimen of material 4 in. by 10 in. in size shall show no signs of disintegration after 10 cycles of freezing and thawing. Hairline cracks shall not be taken as signs of disintegration.

(3) Resiliency. A specimen 4 in. by 5 in. in size shall be compressed at the rate of 1/10 in. per min. to 50 per cent of its original thickness, then the load released, this compression being repeated five times, and after the fifth compression the specimen shall return to at least 70 per cent of its original thickness within one hour after being released from last compression.

(4) Extrusion. A specimen 4 in. by 5 in., confined on three sides, when compressed to half its original thickness shall not extrude more than 25 per cent of its original thickness.

(5) Loss of Asphaltic Saturant. A specimen, when compressed under Clause (3) for resiliency, shall not lose more of the asphaltic compound than 2 per cent by weight of the original specimen tested.

(6) Compressibility. A specimen, when tested for resiliency, shall not require a pressure in excess of 500 lb. per sq. in.

Numerous tests, both of an investigational nature and for acceptance purposes, were run on samples of this material. Representative results of compression and chemical tests are given in Table 32.

It will be noted that the material showed no extrusion when compressed to 50 per cent of its original thickness, and met the requirement for compressibility, requiring a load of less than 500 lb. per sq. in.

TABLE 32
REPRESENTATIVE RESULTS OF PHYSICAL AND CHEMICAL TESTS ON BITUMINOUS PREMOLDED FIBER JOINT MATERIAL
(Illinois Division of Highways Tests)

Item	Sample Number											
	1	2	3	4	5	6	7	8	9	10	11	12
Bitumen, percentage	37.7	41.6	43.1	37.7	44.4	38.2	44.2	41.8	37.4	39.2	51.1	54.6
Fiber, percentage	62.3	58.4	56.9	62.3	55.6	61.8	55.8	58.2	62.6	60.8	48.9	45.4
Original thickness, in.	0.776	0.752	0.714	0.732	0.742	0.722	0.792	0.794	0.802	0.805	0.754	0.756
Recovery, percentage of original thickness	73.1	75.2	73.6	73.8	72.9	74.4	74.3	74.3	72.0	73.6	74.2	73.4
Load, lb. per sq. in. for 50 per cent compression	441	439	492	441	499	481	124	135	316	486	483	496
Percentage of bitumen loss after 50 per cent compression	None	None	None	None	None	None	None	None	None	None	Trace	Trace
Extrusion, percentage of original thickness	None	None	None	None	None	None	None	None	None	None	None	None

to compress it to half its original thickness, and the requirements for asphaltic content. The samples passed the requirement for resiliency, having recovered to more than 70 per cent of their original thickness one hour after the fifth compression. Samples invariably passed the loss of asphaltic saturant test, except when the material was shipped from the factory before it was thoroughly cured and dried.

Results of freezing and thawing tests on representative samples of fiber board are given in Table 33. While the samples did not fail in 10 cycles of freezing and thawing by cracking or splitting, they did become very soft and had increased in thickness 10 to 15 per cent. In actual service a joint in this condition would take in solid material from the surface of the road, which later on would increase resistance to compression when the joint closed and reduced the space for expansion. Furthermore, the softening indicates that the asphalt is not so efficient in waterproofing as claimed by the manufacturers. The material is highly susceptible to the absorption of water in large quantities, probably because of the large number of void spaces contained in it. Thus it may be expected, especially in the case of unsealed joints, that the joints will collect and hold water which may later drain into the soil and soften the subgrade immediately adjacent.

TABLE 33
RESULTS OF FREEZING AND THAWING TESTS ON BITUMINOUS PREMOLDED FIBER JOINT MATERIAL
(Illinois Division of Highways Tests)

Item	Sample Number						
	1	2	3	4	5	6	7
Original thickness, in.	½	½	1	1	½	½	½
Dimensions, in.	4 x 10	4 x 10	4 x 10	4 x 10	4 x 10	6 x 12	6 x 12
Wet weight at end of 10 cycles freezing and thawing, gr.	349	361	586	625	310	514	543
Original dry weight, gr.	146	147	283	294	151	267	267
Weight of water absorbed, gr.	203	214	303	331	159	247	276
Percentage of absorption	139.0	145.6	107.1	112.6	105.3	92.5	103.4

IV. Field Investigations

11. *Résumé of Investigations Prior to 1937.*—From the time metal joints were first adopted in 1933, their installation and subsequent operation were carefully observed. From time to time, as experience dictated, changes in construction practices were made to improve installation, and manufacturers were required to correct defects which became apparent in the joints.

The constant surveillance to which joints were subjected disclosed that the J-2 joint installed during 1934 had a number of serious defects resulting in faulty installations. This led to the first major field investigation of joints, started late in 1934 and finished in the spring of 1935. Representative joints were critically examined on every section of pavement built in 1934 in which J-2 joints had been installed. Defects discovered included lack of lateral and vertical stiffness; collapse of side walls with resultant decrease in expansion space; large areas of honeycomb adjacent to the joints, especially around the lugs on the load transmission device; failure of the joint to conform with the subgrade; the wires which held the copper seal in place tore the flanges and bent them out of shape; it was difficult to compact concrete under the flange of the copper seal; and the splice between the two pieces of copper seal would not stay in place. These defects were so prevalent and serious that the use of the J-2 joint was discontinued.

Fairly extensive investigations were also made of the J-1 and J-4 joints, especially those in the pavements constructed in 1935 in which the defects found in earlier joints had been corrected. The investigation was continued on pavements constructed in 1936, primarily to secure data which would be of value to the construction organization in improving the method of installation of the joints. From this investigation it appeared that good results were being obtained generally. However, there were indications that the joint spacing was too long for efficient control of transverse cracking, some intermediate cracking being in evidence.

12. *Investigation by University of Illinois Committee.*—In 1937, the University committee, whose appointment has been already discussed, undertook as a part of its study of the expansion joint problem an extensive field investigation. The investigation included a thorough examination of a large number of joints in Illinois and in four eastern states, five southern states, six midwestern states, exclusive of Illinois, one western state, and the District of Columbia.

The field investigation consisted of careful examinations of the joints themselves, the condition of the adjacent concrete, and the general condition and amount of cracking of the slabs. The condition of the bituminous caps on metal joints was observed, and the caps removed so that at least a portion of the copper seal of each joint could be examined for splits and other defects. Holes were cut in many of the copper seals, and the interior of the joint examined to see if water and dirt had entered. The end seals on all joints were examined, and in many cases they were removed to permit a more thorough examination of the interior of the joint. The concrete faces adjacent to the joints, especially around the load transmission devices, were examined for honeycombing and other evidence of improperly compacted concrete. More than a thousand joints were examined in detail, and many more were examined for specific defects. Highway engineers for the state in which the examinations were made usually accompanied the member or members of the committee making the survey. In Illinois, engineers from the office of the District Engineer of the district in which the construction was located acted as guides for the committee.

13. *Investigation by Illinois Division of Highways — 1937.* — This investigation, ordered by the Chief Highway Engineer, was conducted by engineers from each district under the direction of the Bureau of Materials. It included a general examination of 657 expansion joints on 65 different paving sections. Each district examined at least 10 joints of each type for each year they were installed from 1933 to 1936, inclusive. These included 144, 90, 92, and 96 J-1 joints installed during 1933, 1934, 1935, and 1936, respectively; 60 and 73 J-4 joints installed during 1935 and 1936, respectively; and 102 J-2 joints installed during 1934 and 1935. In addition, the concrete adjacent to the ends of 373 other J-2 joints was examined for a defect which had been found to be prevalent on some sections containing this type of joint. Also, when it was discovered in one district that the copper seals had split, 262 joints of all types were examined to determine whether this failure was universal.

The general procedure followed was to examine the joints selected on each section and the pavement adjacent thereto for evidence of inadequate load transmission; displaced end plates; concrete failures at the corners of the slabs; spalling along the joints; alignment; condition of copper seals; condition of bituminous cap; collapse of the side walls at the ends of the joints; evidence that the joint was or was not free to move; and the amount of maintenance the joints had been given. Also, the copper seal was removed from two or more of the joints on each section to permit an examination of the interior for evidence of collapse, dirt, and water. Each district selected the sections on which joints were to be examined and furnished the personnel to make the survey. The data collected were submitted to the Bureau of Materials for analysis.

In the case of supplemental surveys to determine the extent of the splitting of copper seals on all types of joints, and of the cracking of the concrete adjacent to the ends of J-2 joints, the work was done by engineers from the Bureau of Materials in company with representatives from the district in which the pavement was located.

14. *Investigation by Illinois Division of Highways — 1939.* — This investigation was initiated in November, 1939, by the Director of the Department of Public Works and Buildings for the purpose of determining beyond any question of doubt whether metal joints of the type used from 1932 to 1938 are or are not suitable for use. In planning an outline of procedure for the investigation, those who had served on the University of Illinois committee in 1937 were consulted, and many suggestions based on their knowledge of the problem and their previous experience were incorporated in the final outline.

It was decided that the examinations should be made by district personnel and the results analyzed by the Bureau of Materials. At a meeting held November 20, 1939, the problem and plans were discussed in detail with the District Engineers. A few days later, mimeographed copies of the final instructions were sent to each district, so that each man engaged in the work would be thoroughly familiar with the procedure to be followed.

The program included the examination of 562 J-1 expansion joints and 570 J-1 contraction joints installed about equally during 1935, 1936, and 1937; 228 each of J-2 expansion and contraction joints selected equally from those installed during 1934 and 1935, the only years these joints were used; 187 each of J-4 expansion and contraction joints selected equally from those installed during 1935, 1936, and 1937; 328 bituminous premolded fiber expansion joints installed during 1938 and 1939; 242 4-in. open expansion joints installed from 1923 to 1935; and 50 bituminous premolded expansion joints installed during 1933 and 1938.

The features to be observed and the particular defects to be looked for were stated definitely in the instructions sent to the districts; detailed instructions as to exact procedure were not given. It was thought best to leave the organization of the work and the methods to be employed to the districts, so that they could make their plans to fit the personnel available and the equipment on hand.

The investigation included an examination of each joint for the following features and defects:

(a) Measurement of elevations at various points on the pavement surface adjacent to the joint and of the filler over the joint, to determine whether the presence of the joint influenced the contour of the surface.

(b) Spalling of the edges of the concrete adjacent to the joint.

(c) The condition of the bituminous premolded caps on the J-1 joints and of the poured bituminous filler on the J-2 and J-4 joints.

(d) Splits or fractures in the copper top seals.

(e) Type of material in the shoulders, and shoulder conditions which might have affected the joints.

(f) Splits or fractures in the copper end seals, and corroded end plates.

(g) Presence of water, ice, and dirt inside the joint, and available expansion space.

(h) Fractures in edge of pavement adjacent to ends of joint.

(i) Type of load transmission device used and evidence of inadequate load transmission.

(j) Unusual subgrade, drainage, and other conditions which might have affected the operation of the joint.

(k) The condition of the premolded joint filler, width of joint opening, space between filler and concrete, and amount of dirt in this space.

(l) The width of the poured joints; whether an original, a cut, or a recut joint; failure of concrete adjacent thereto; and condition and effectiveness of poured filler.

(m) The total number of transverse cracks that had occurred between joints in every section on which joints were examined, and such features as culverts, bridges, cuts, fills, grade, and other conditions which might have influenced the formation of cracks being observed.

Supplementing this investigation, the fiber joints in 18 of the sections included in the 1939 investigation were re-examined in the spring of 1943 to obtain more information regarding the effect of age on this type of joint, and to determine whether the wire mesh reinforcement which these pavements contained was still effective in holding transverse cracks tightly closed. In 1945 another survey was made of the pavements built with wire mesh reinforcement to check further the effectiveness of the wire mesh.

15. *Results of Investigations.*— The results of the various field investigations are generally in good agreement. For a number of reasons, notably because the investigations were more extensive and perhaps better planned, and because the investigators had benefited through the experience obtained from previous surveys, the University of Illinois committe investigation of 1937 and the investigations conducted by the Division of Highways in 1937 and 1939 produced by far the greatest volume of specific information. The results of earlier surveys showed general trends, but the later surveys yielded results which permitted more definite conclusions to be drawn. In presenting the results, all the investigations are considered, but specific information is drawn largely from the three investigations mentioned above.

(a) Condition of Copper Seals

Perhaps the most serious defect shown by the investigations is the splitting and cracking of the copper seals on metal-sealed joints. Failures of this kind were first found in 1937, independently by the Division of Highways and the University committee. At that time the University committee reported it had found extensive failures of the copper seals and recommended that the use of copper-sealed joints be discontinued. Almost half of the 262 expansion joints examined specifically for these failures by engineers from the Division of Highways had splits in the copper seals. Both investigations indicated that the failures were influenced by traffic, more extensive failures occurring on heavily traveled pavements.

In the 1939 investigation conducted by the Division of Highways, not only the number of joints with split seals was observed but also the length of the failures was measured. Other conditions, such as bent seals and foreign material on the seal which might induce and aggravate failures, were observed. Summaries of the results are given in Tables 34 to 37, inclusive. Of principal interest in these tables are the average lengths of fracture at each of the vulnerable points of the seals, indicated by the numbers on the sketches below the tables, and the total average length of fractures, which is the sum of the average lengths at all points. The average values were obtained by dividing the total length of fractures by the total number of joints examined. The contraction joint seals were difficult to examine, especially where the joints were not edged, and it was not possible to examine all the contraction joints included in the survey. Since the number examined was not clearly shown by the field reports, the averages, based on the total number included in the survey, are probably somewhat low. This difficulty was not encountered with the expansion joints.

In discussing these results, it should be pointed out that the values given in the tables do not represent progressive developments on the same joints but are the results of measurements on separate groups of joints of the given ages. This explains what may appear in some cases to the casual reader as retrogression.

The tables bear out what all the investigations indicated — that splitting of copper seals is a function of age. Table 34 shows that on the average the length of failures in the seals of J-1 expansion joints amounted to 0.42 ft. at two years, 1.55 ft. at three years, and 0.94 ft. at four years. Similar values for the four- and five-year-old J-2 expansion joints were 7.03 ft. and 4.57 ft., respectively. The J-4 expansion joint, possibly because the construction of the seal was such that once closed, subsequent movements were generally taken by one of its three folds, developed failures much earlier than the other seals. For this joint, the average total length of failures per joint two, three, and four years old was 6.78 ft., 14.97 ft., and 20.84 ft., respectively.

In all the investigations, it was found that the amount of splitting in all types of contraction joint seals was much more uniform than in the case of expansion joints. This is shown by the numerical results for the contraction joints examined during the 1939 investigation, given in Table 35. The average total splits in seals of J-1 contraction joints two, three, and four years old were 0.42 ft., 0.76 ft., and 2.67 ft., respectively; for the four- and five-year old J-2 contraction joints 4.03 ft. and 1.90 ft.; and for J-4 contraction joints two, three, and four years old, 0.53 ft., 0.60 ft., and 1.85 ft., respectively.

It will be noted that there was little difference between the results for the J-1 and J-4 contraction joint seals, as should be expected, since the two seals were almost identical in design. There was somewhat greater splitting of the J-2 contraction joint seals at four years. The design of this seal, shown by the sketch below Table 35, was such that solid matter could be packed into the U-shaped trough, a condition which appeared to be conducive to the type of failure found in these seals. It should be remembered that many of the contraction joint seals could not be examined because the joints were tightly closed, and the averages, being computed on the basis of the number of joints included in the survey and not on the number actually examined, are undoubtedly low.

The percentage of joints included in the 1939 investigation having fractured top seals, and the length of the fractures in percentage of the total length of seals examined, are shown in Tables 36 and 37 for expansion and contraction joints, respectively. The data, arranged by

TABLE 34
SUMMARY OF DATA RELATING TO CONDITION OF COPPER TOP AND END SEALS ON METAL EXPANSION JOINTS
(Survey made during winter of 1939-40)

Age years	Type of Joint	Number of Joints	Copper Top Seal: Number bent, percentage: Total	With traffic	Against traffic	Average length of fracture, ft.: Location 1	2	3	4	5	Total	Joints with foreign material adjacent to seal, percentage	Fractures caused by foreign material, percentage	Number of joints set too high, percentage	End Seals: Fractured, percentage	Average length of fracture, in.
2	J-1	201	11.4	7.9	4.9	0.02	0.10	0.21	0.05	0.04	0.42	80.0	2.5	9.5	31.6	1.8
	J-4	77	16.9	16.9	15.6	....	0.91	4.59	1.28	....	6.78	94.8	29.9	6.5	16.9	0.6
3	J-1	166	42.5	25.0	11.4	0.02	0.19	1.09	0.22	0.03	1.55	98.1	15.7	9.6	27.7	1.1
	J-4	70	37.1	24.3	....	0.04	4.85	4.07	5.95	0.06	14.97	97.1	31.4	1.4	7.1	0.2
4	J-1	199	43.6	29.5	14.1	0.03	0.38	0.23	0.25	0.05	0.94	93.8	23.1	5.1	25.6	0.4
	J-2	100	1.0	1.0	....	....	0.14	6.86	0.03	....	7.03	97.0	40.0	11.0	....	...
	J-4	40	47.5	45.0	25.0	....	5.33	1.23	7.83	6.25	20.84	85.0	32.5	2.5	16.3	0.3
5	J-2	128	7.8	6.3	6.3	....	0.24	4.29	0.03	0.01	4.57	86.9	36.7	6.6	....	...

NOTE: 95 and 62 per cent of the galvanized end sleeves on the 4- and 5-year old J-2 joints, respectively, were rusted.

Location of Fracture

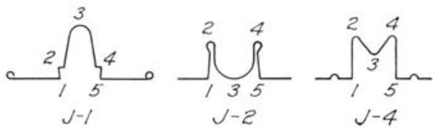


TABLE 35
SUMMARY OF DATA RELATING TO CONDITION OF COPPER TOP SEALS AND GALVANIZED END PLATES ON METAL CONTRACTION JOINTS
(Survey made during winter of 1939-40)

Age years	Type of Joint	Number of Joints	Copper Top Seal: Number bent, percentage: Total	With traffic	Against traffic	Average length of fractures, ft.: Location 1	2	3	4	5	Total	Joints with foreign material adjacent to seal, percentage	Fractures caused by foreign material, percentage	Number of joints set too high, percentage	Fracture caused by contraction plates, percentage: Yes	No	End Seals: Condition of end plates, percentage: Good	Rusty
2	J-1	209	17.2	4.3	3.3	....	0.42	....	....	....	0.42	70.3	0.0	18.7	...	84.2	77.9	13.1
	J-4	77	3.9	1.3	...	....	0.53	....	....	....	0.53	92.2	3.9	10.4	...	100.0	70.1	17.5
3	J-1	166	24.0	10.5	11.1	0.04	0.72	....	....	....	0.76	70.4	1.2	21.1	0.6	71.0	57.9	13.1
	J-4	70	8.6	2.8	...	....	0.60	....	....	....	0.60	30.0	11.4	7.1	...	67.1	74.3	15.7
4	J-1	195	20.4	3.9	4.6	....	2.51	0.16	....	....	2.67	46.7	9.7	27.2	...	73.8	50.5	30.5
	J-2	100	7.0	5.0	3.0	....	4.02	0.01	....	....	4.03	40.0	4.0	4.0	...	56.0	76.0	19.9
	J-4	40	10.0	2.5	2.5	....	1.85	....	....	....	1.85	40.0	2.5	....	2.5	55.0	75.0	25.0
5	J-2	128	10.9	4.7	2.3	....	1.12	0.18	0.60	....	1.90	89.8	14.8	11.3	3.9	79.7	60.2	24.2

Location of Fracture

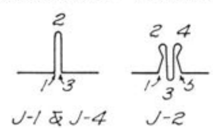


TABLE 36
SUMMARY SHOWING THE EXTENT OF FAILURES IN COPPER TOP SEALS ON METAL EXPANSION JOINTS
(Survey made during winter of 1939-40)

Age years	Type of Joint	Number of Joints Examined	Joints with Fractured Seals		Total Length of Seals Examined ft.	Total Length of Fractures	
			Number	Per-centage		Ft.	Per-centage
2	J-1 J-4	201 77	34 43	16.9 55.8	4,058 1,554	84.87 522.40	2.1 33.6
Totals and Averages		278	77	27.7	5,612	607.27	10.8
3	J-1 J-4	156 70	78 68	50.0 97.1	3,296 1,360	258.22 1,066.05	6.8 78.4
Totals and Averages		226	146	64.6	4,656	1,324.27	28.5
4	J-1 J-2 J-4	195 100 40	82 83 36	42.1 83.0 90.0	3,840 1,920 800	183.58 702.62 576.33	4.8 36.6 72.0
Totals and Averages		335	201	60.0	6,560	1,462.53	22.3
5	J-2	118	84	71.2	2,400	602.00	25.1

type of joint and age of pavement, show clearly the serious extent of failures in the copper top seals. Even the J-1 expansion joint, whose seal was the least susceptible to splitting, had 17 per cent failures at two years, 50 per cent at three years, and 42 per cent at four years.

TABLE 37
SUMMARY SHOWING THE EXTENT OF FAILURES IN COPPER TOP SEALS ON METAL CONTRACTION JOINTS
(Survey made during winter of 1939-40)

Age years	Type of Joint	Number of Joints Examined	Joints with Fractured Seals		Total Length of Seals Examined ft.	Total Length of Fractures	
			Number	Per-centage		Ft.	Per-centage
2	J-1 J-4	209 77	23 21	11.0 27.3	4,022 1,554	87.60 40.95	2.2 2.6
Totals and Averages		286	44	15.4	5,576	128.55	2.3
3	J-1 J-4	156 50	25 17	16.0 34.0	3,296 1,160	125.98 41.98	3.8 3.6
Totals and Averages		206	42	20.4	4,356	167.96	3.8
4	J-1 J-2 J-4	195 100 40	74 26 11	37.9 26.0 27.5	3,840 1,920 800	520.20 202.90 74.10	13.6 10.6 9.3
Totals and Averages		335	111	33.1	6,560	797.20	12.1
5	J-2	118	50	42.4	2,400	275.12	11.5

The J-2 expansion joints had 83 per cent failures at four years and 71 per cent at five years. The J-4 expansion joints had 56 per cent failures at two years, 97 per cent at three years, and 90 per cent at four years. As already mentioned, the high proportion of failures in the J-4 joint probably was due to the design of its seal.

The length of fractures in percentage of the total length of seal agrees in general with the percentage of joints with split seals. The splits in the J-1 expansion joint seals amount to 2 per cent of the total length of seals at two years, 7 per cent at three years, and 5 per cent at four years. The four- and five-year-old J-2 expansion joints had fractures aggregating 37 and 25 per cent, respectively. For the two-, three-, and four-year-old J-4 expansion joints, the splits in the seals amounted to 34, 78, and 72 per cent of the total length, respectively.

In the case of the contraction joints included in the 1939 investigation, the increase in fractures with age is shown in Table 37 to be even more definite than with the expansion joints. The joints with fractured seals amounted to 15 per cent of those two years old, 20 per cent of those three years old, 33 per cent of those four years old, and 42 per cent of those five years old. While a larger percentage of J-4 than J-1 contraction joint seals were split at two and three years, the reverse was true of the four-year-old joints, and it is probable that on the average one seal is no better than the other, especially since both are similar in design. This is borne out by the agreement in the total length of fractures for the various types of joints. At two years the fractures in the J-1 and J-4 contraction joint seals amounted to 2.2 and 2.6 per cent, respectively, of the total lengths of each examined; at three years the respective amounts were 3.8 per cent and 3.6 per cent; and at four years 13.6 and 9.3 per cent. It should be noted that the total length of fracture is much lower for contraction joints than for expansion joints, indicating that the forces which cause the copper seal to split, whatever they may be, have less effect on the contraction joint seals. However, as already explained, had it been possible to examine all the contraction joints, the average total length of fractures no doubt would be greater than that shown in Table 37.

Every engineer who was engaged in the several field investigations was convinced that the early and extensive failures of the copper seals were the result of a combination of a number of destructive forces. It is certain, in view of the laboratory tests, that the seasonal opening and closing of the joint, due to contraction and expansion of the concrete, by itself could not have caused the failures in so short a time. Observations indicate that the first closing of the joint pinched the seal into a shape which was more easily affected by subsequent movements and other forces. The causes for failure of the seals are concisely stated in the following excerpt from the official report of the University committee:

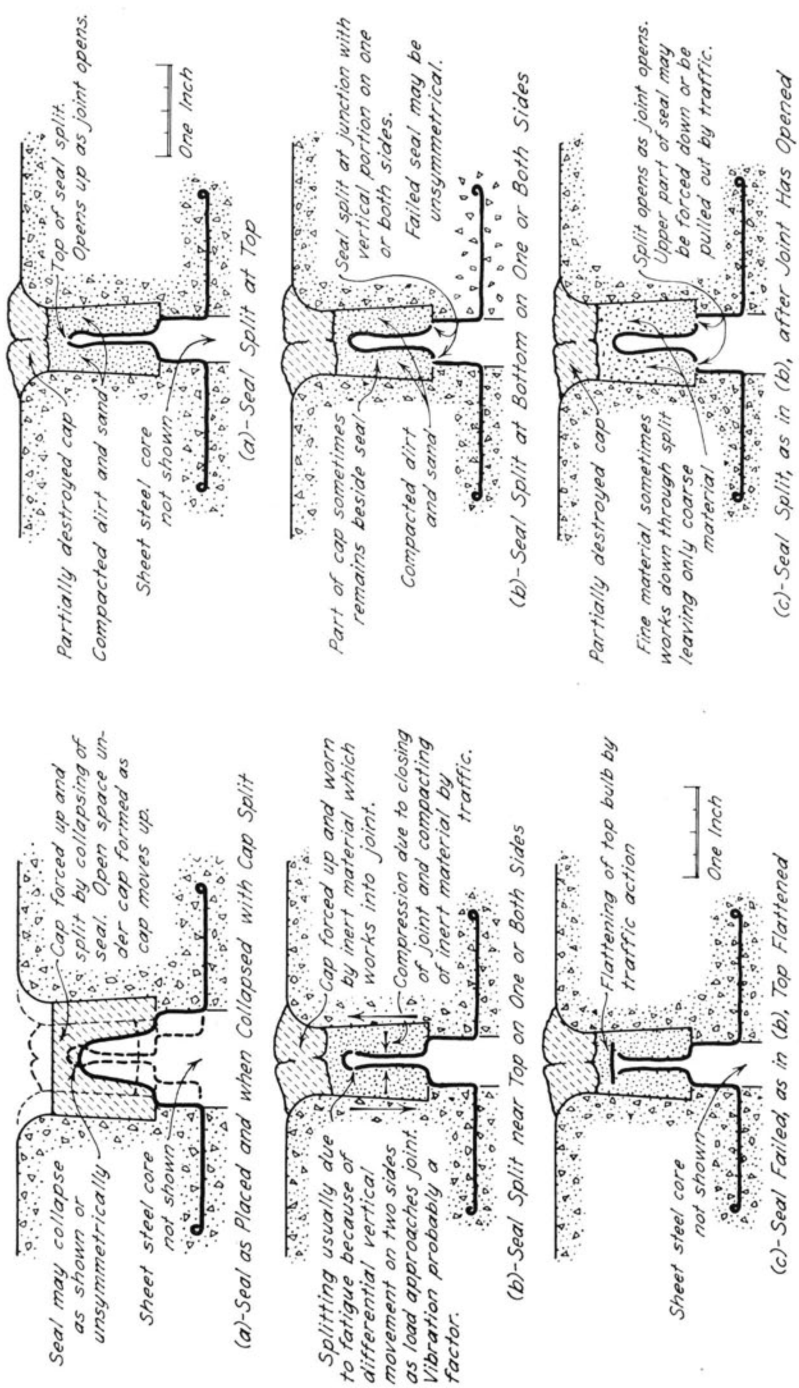
"An important factor which contributes to the failure of the copper seal of a joint of the type called for by the present specifications[9] appears to be the vertical force which is exerted on the seal by traffic pounding on the cap and the inert material which works into the joint above the seal. Another important factor appears to be the differential vertical movement between the transverse edge of the loaded slab and the adjacent transverse edge of the unloaded slab as a load passes over the joint. Added to the latter effect may be vibration at the ends of both slabs due to the passing loads. Fatigue appears to be an important factor in the failure of copper seals at joints."

That traffic contributed to pinching and bending of seals on expansion joints is shown by data from the 1939 investigation given in Tables 34 and 35. Bent seals were found on from 1 to 47.5 per cent of the expansion joints, and the fact that generally more were bent in the direction of traffic than in the opposite direction indicates that traffic was the cause. This is also borne out by the increase in bending with age.

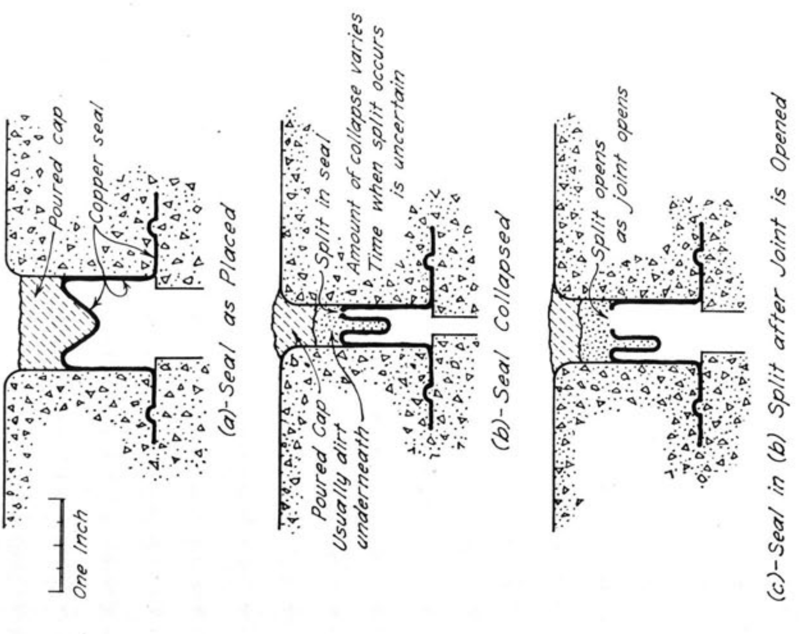
The effect of pinching of the seal due to closing of the joint, and the action of traffic in contributing to the splitting of copper seals on the various types of expansion joints, as observed from the extensive examinations made in the field, are shown diagrammatically by sketches in Figs. 63 to 66, inclusive. Figures 63 and 64 show that the seal on the J-1 expansion joint may fail in one or more of three different ways, all starting after the top of the seal becomes tightly pinched and loses a great deal of its freedom from restraint. After the seal assumes the position shown by the dotted lines in Fig. 63a, it may split at each side of the crimped top as shown in Fig. 63b, following which the top of the bulb will be flattened by the pounding of traffic (Fig. 63c). At other times the failure may occur at the top of the crimp as shown in Fig. 64a. Failure may also develop in the sharp bends where the seal rests on the sheet steel stool as shown in Fig. 64b. Figure 64c shows what happens when the joint opens after the seal has split as illustrated in Fig. 64b.

Figure 65 illustrates the development of a typical failure in a J-4 seal. The joint closes from the initial position shown in Fig. 65a to that shown in Fig. 65b, where the folds of the seal are tightly pinched. Subsequent movements of the joint develop high strains in the vicinity of one or more of the pinched folds, finally resulting in the copper

[9] Standard Specifications in effect in 1937.



FIGS. 63 AND 64. TYPICAL FAILURES OF J-1 EXPANSION JOINT SEALS IN SERVICE



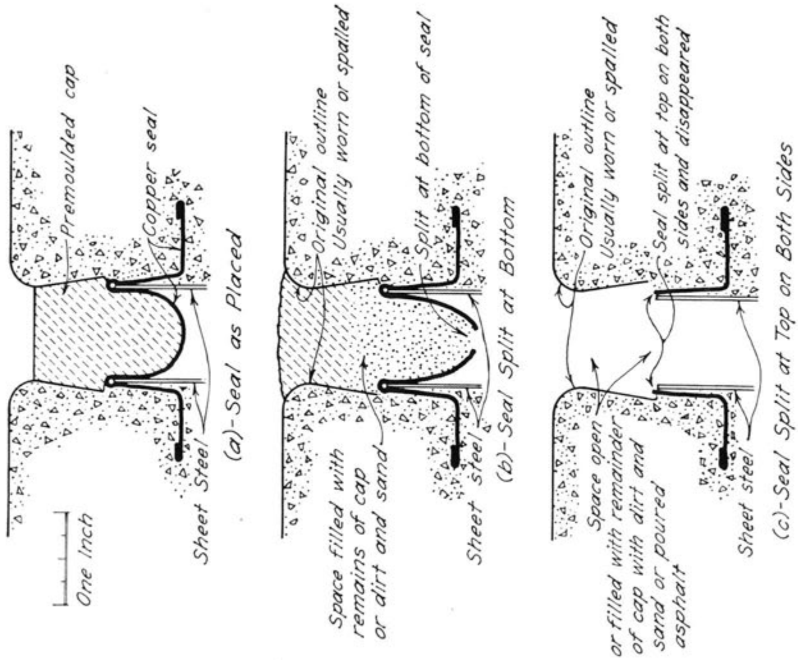


FIG. 65 (AT LEFT). TYPICAL FAILURE OF J-4 EXPANSION JOINT SEALS IN SERVICE
FIG. 66. TYPICAL FAILURES OF J-2 EXPANSION JOINT SEALS IN SERVICE

splitting as shown in Fig. 65b. The failure is further aggravated when the joint opens, as shown in Fig. 65c.

Figure 66 shows two typical failures which occur in the seal on the J-2 joint. The seal may split in the bottom of the trough, as shown in Fig. 66b, due to the vertical forces of wheel loads being transmitted through the bituminous filler and the dirt which accumulates above the seal; or the seal may tear along the folds where it passes over the sheet steel side walls of the joint, as shown in Fig. 66c.

The investigation conducted by the University committee and that made in 1937 by the Division of Highways further indicated that the amount of traffic influenced failure of the seals; that is, more failures occurred on pavements carrying heavier traffic. Analysis of the data from the 1939 investigation does not bear this out. The proportionate length of failures was as great on pavement carrying 100 to 1,000 total vehicles per day as it was on pavements carrying 1,000 to 6,000 total vehicles per day, totals including both passenger and commercial vehicles. Neither did a comparison based on commercial vehicles alone reveal the existence of any relation between failure of seals and amount of traffic. The length of fractures in expansion joint seals under light, medium, and heavy commercial traffic, respectively, amounted to 23.8, 26.1, and 20.8 per cent of the total length of seal examined, considering light traffic as 0 to 200, medium traffic as 200 to 500, and heavy traffic as 500 to 1,500 commercial vehicles per day. Regardless of this analysis, observations made of the action at joints as vehicles pass over them, particularly the heavier commercial vehicles, lead to the belief that heavy traffic influences the splitting of copper seals. This belief is substantiated by the fact that failures invariably begin at the normal wheel paths.

In all the investigations, dirt, sand, and other foreign material were found on top of the copper seals on a great majority of the joints examined. This material worked down under the cap, either poured or premolded, and was deposited on the seal. Being highly incompressible, it restricts the freedom of the seal to close so that the copper becomes tightly pinched, a condition which predisposes the seal to early failure. Furthermore, vertical forces produced by heavy wheel loads passing over the joint are transmitted through the incompressible material to the seal with little or no cushioning.

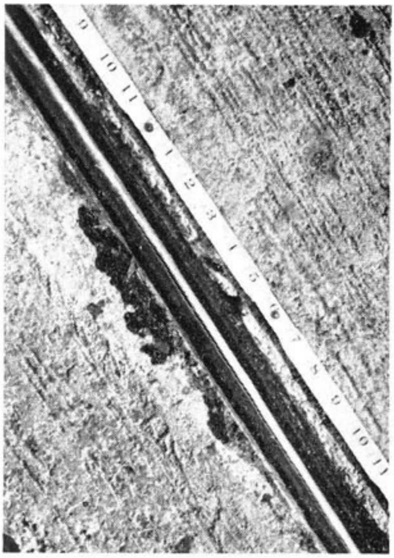
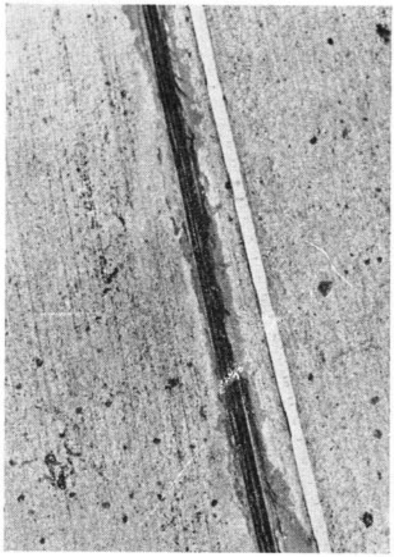
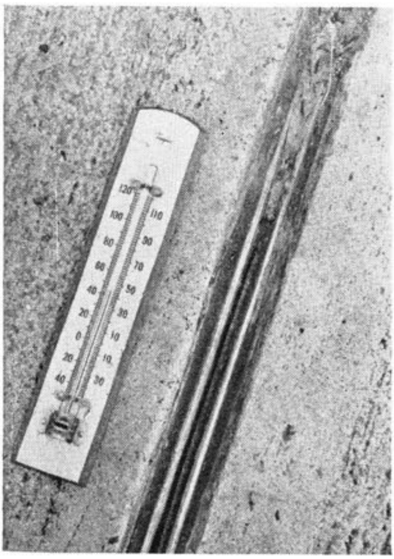
Table 34 shows that 80 to 98 per cent of the expansion joints examined during the 1939 investigation had foreign material on top of the seals. The engineers who made the examinations were of the opinion that this condition was the cause for many of the split seals. They believed that this condition was a somewhat greater factor in the J-2 and J-4 joints, because the shape of the seal is such that foreign material becomes packed in the trough causing severe strains in the copper. Table 35 shows that 30 to 92 per cent of the contraction joints had foreign material adjacent to the seal; however, this was not considered a major cause for fractures, since the joint opening was so narrow that vertical forces from wheel loads probably were not transferred to the seal.

In the course of the 1939 investigation, a study was made to determine how many joints had been set high; that is, with the seal closer to the pavement surface than required by the plans, it being thought that such a condition might be responsible for failure of the seals. It was found that 1.4 to 11 per cent of the expansion joints had been set high. The proportion of high contraction joints was somewhat greater, ranging from 4.0 to 27.2 per cent. In neither case is there any relation between the percentage of high joints and the extent of failure, and it may be concluded that the manner in which these joints were set had no marked influence on the failure of copper seals. However, in a few instances, particularly in the case of contraction joints, where the joints were set so high that the wheels of vehicles were actually in contact with the seals, field reports showed that excessive failures occurred.

The possibility that failure of seals on contraction joints was caused by the copper being stretched tightly over the top edge of the dividing plate as the joint opened, was considered in the several investigations. All the investigators concluded that there was little or no evidence to indicate that this was a factor contributing to the failure of contraction joint seals.

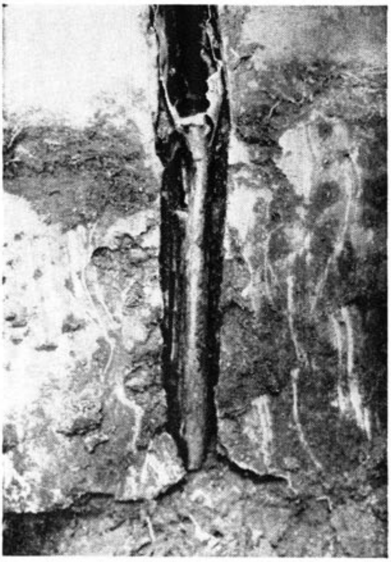
Copper end seals were somewhat less susceptible to failure than the top seals. Table 34 shows that from 7 to 32 per cent of the J-1 and J-4 expansion joints examined during 1939 had split end seals. For some unknown reason, a greater proportion of J-1 end seals were fractured, and the average fractures were somewhat longer, than in the case of the J-4 joints. The J-2 expansion joints did not have copper end seals, being covered by a steel plate fitted with a galvanized steel sleeve. Of the four- and five-year-old joints examined, 95 and 62 per cent, respectively, had rusted end sleeves. As shown by Table 35, a large percentage of the galvanized steel end plates on the contraction joints were in good condition, only 13.1 to 30.5 per cent being reported as corroded.

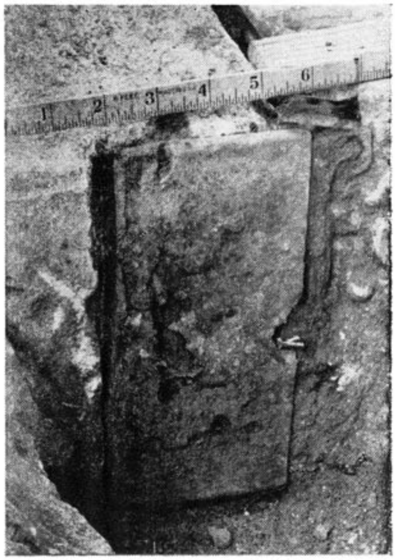
Typical examples of failures in top seals and end seals are shown in Figs. 67 to 73, inclusive. Figure 67 shows fractures in the top seal of a J-1 expansion joint. Figure 68 is typical of the failures in J-2


FIGS. 67, 68 (ABOVE), 69, 70 (BELOW). TYPICAL EXAMPLES OF FRACTURED TOP SEALS — RESPECTIVELY ON J-1, J-2, AND J-4 EXPANSION JOINTS AND ON J-2 CONTRACTION JOINTS (LATE DESIGN)

expansion joint top seals; Fig. 69 illustrates a condition prevalent in top seals of J-4 joints, while Fig. 70 shows a typical failure of a top seal on a J-2 contraction joint. The J-1, J-4, and to some extent the





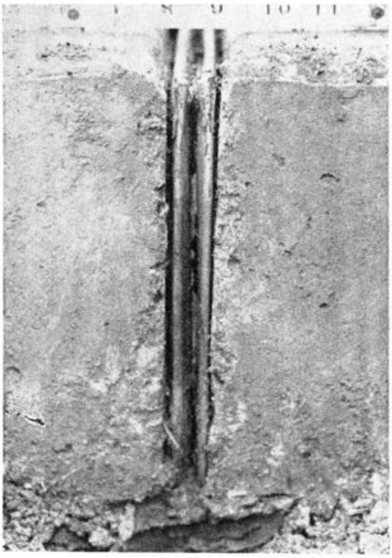


FIG. 71 (ABOVE, AT LEFT). TYPICAL EXAMPLE OF FRACTURED END SEAL ON J-1 EXPANSION JOINT

FIG. 72 (ABOVE, AT RIGHT). TYPICAL CONDITION OF GALVANIZED END SLEEVE ON J-2 EXPANSION JOINT

FIG. 73 (AT LEFT). TYPICAL EXAMPLE OF FRACTURED END SEAL ON J-4 EXPANSION JOINT

J-2 contraction joints used a seal shaped as shown in the photograph, and the failure is typical for a top seal of this design. A split end seal on a J-1 expansion joint is shown in Fig. 71. Figure 72 is typical of the condition of the galvanized end sleeves on J-2 expansion joints, and a typical example of the failure of end seals on J-4 expansion joints is shown in Fig. 73.

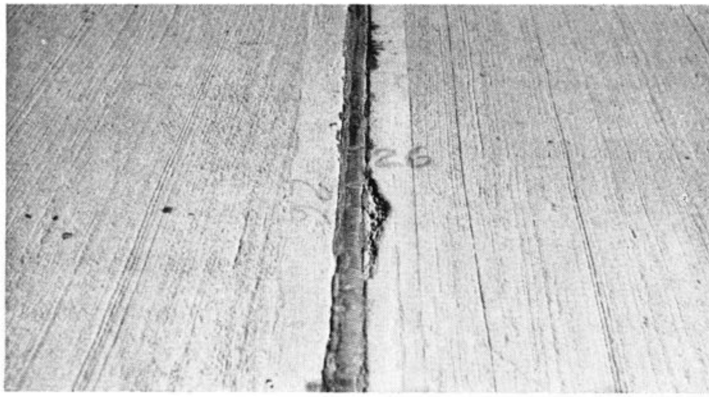
(b) Spalling at Joints

Spalling, as used in this bulletin, refers to the breaking off of the upper edge of the concrete slab adjacent to joints or cracks, or at the junction of transverse joints or cracks and longitudinal joints. Spalling results from a number of causes: principally, wheel loads; honeycomb around the joint; compression when the joint closes, especially when incompressible foreign material is present in the top of the joint; horizontal planes of weakness formed by the flanges of the metal seals; and the fact that in placing and finishing the concrete it is difficult to avoid having inferior concrete at the joints. Spalling is a progressive failure because, as the concrete breaks, the increased width of the broken surface adds to the intensity of the impact and pounding of wheel loads, factors that produce spalling. Spalling detracts from the appearance of a pavement and adds to the cost of its maintenance.

Spalling of various degrees was observed at many of the joints examined during the investigation by the University committee and by the Division of Highways in 1937. In the 1939 investigation and the supplementary investigation of 1943, spalling was classified as slight or bad. Joints at which spalling was shallow and limited in extent, and which did not contribute materially to the roughness of the pavement, were classified as having slight spalling. Spalling which was prevalent along the joint or at the center joint, and had progressed to the stage where it affected riding, was defined as bad. Typical examples of spalling at several types of joints are shown in Figs. 74 to 81, inclusive.

Figure 74 shows slight spalling along a J-1 expansion joint. Bad spalling at the same kind of joint, obviously caused by the honeycomb visible under the flange of the copper seal, is shown in Fig. 75. Much honeycomb was found adjacent to the metal-sealed joints, and this was thought to be one major cause for spalling along these joints. Another cause of spalling at metal-sealed joints is that the flanges of the seal form planes of weakness in a horizontal direction. The relatively thin layer of concrete above the flanges is easily broken by traffic, especially if there is honeycomb under the flange.

Figure 76 shows bad spalling along a J-2 expansion joint, another example of the result of poorly consolidated concrete under the flange of the seal. A case of slight spalling along a J-4 expansion joint is shown in Fig. 77. Figure 78 shows bad spalling along a J-4 contraction joint, caused by improper compaction of the concrete. A typical example of slight spalling along a 4-in. open joint is shown in Fig. 79.



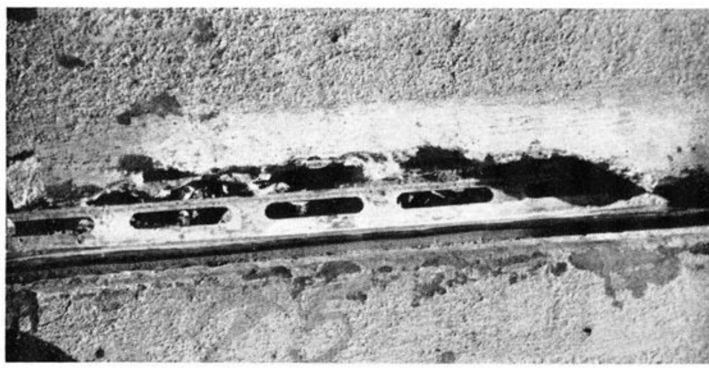


FIG. 74 (AT TOP). SLIGHT SPALLING ALONG J-1 EXPANSION JOINT

FIG. 75 (CENTER). BAD SPALLING ALONG J-1 EXPANSION JOINT

FIG. 76 (AT LEFT). BAD SPALLING ALONG J-2 EXPANSION JOINT

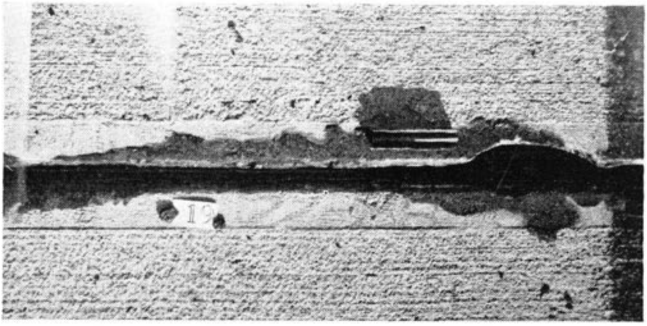


FIG. 77. SLIGHT SPALLING ALONG J-4 EXPANSION JOINT

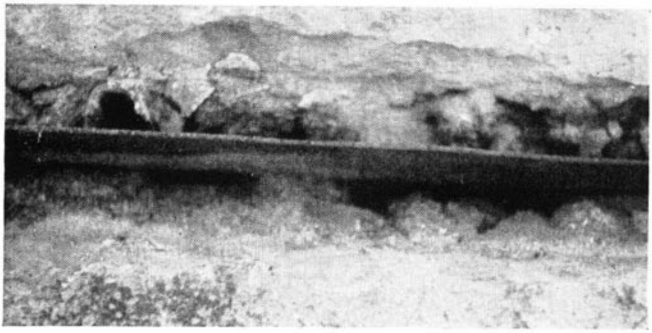


FIG. 78. BAD SPALLING ALONG J-4 CONTRACTION JOINT

Extremely bad spalling along the same type of joint may be seen in Fig. 80. In this joint, almost completely closed, the spalling probably was caused by excessive pressures along the upper edges of the slabs, due to expansion of the concrete. Figure 81 shows typical slight spalling along a fiber joint, a condition which existed at approximately 30 per cent of the fiber joints in the section of pavement on which this joint was located. Only a small amount of bad spalling was found adjacent to fiber joints, probably because the pavements were only one year old when the survey was made.

Tables 38 and 39 are summaries of the data collected during the 1939 investigation, relating to spalling at expansion joints and contraction joints, respectively. They show the percentage of joints of each type and age having no spalling, slight spalling, bad spalling, and the influence on spalling of edging and foreign material in the top of the joint.

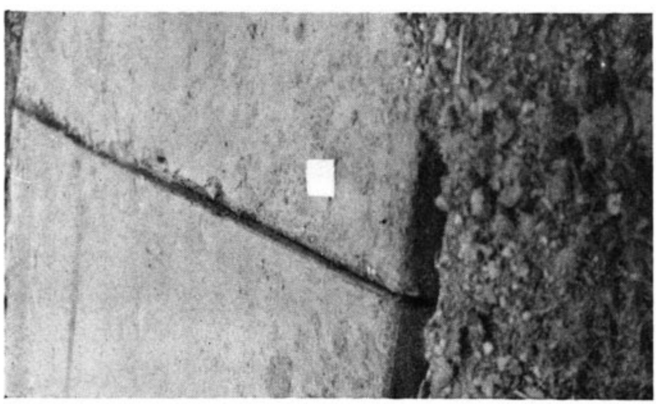


FIG. 79. SLIGHT SPALLING ALONG 4-IN. OPEN JOINT

FIG. 80. EXTREMELY BAD SPALLING ALONG 4-IN. OPEN JOINT

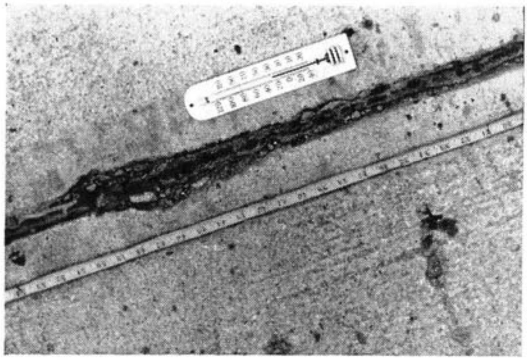


FIG. 81. SLIGHT SPALLING ALONG FIBER EXPANSION JOINT

TABLE 38

SUMMARY OF DATA RELATING TO SPALLING AT EXPANSION JOINTS

(Survey made during winter of 1939-40)[1]

Age years	Type of Joint	Number of Joints	With spalling: Along joint: None	With spalling: Along joint: Slight	With spalling: Along joint: Bad	With spalling: At center junction: None	With spalling: At center junction: Slight	With spalling: At center junction: Bad	With spalling: Caused by dirt in joint: Yes	With spalling: Caused by dirt in joint: No	With spalling: Affected by edging: Yes	With spalling: Affected by edging: No	With joint edged: High	With joint edged: Low	With joint edged: Level	With concrete broken by asphalt
1	Fiber	328	54.6	43.0	2.4	90.5	8.5	1.0	...	....	1.8	85.1	3.4	5.5	73.5	0.3
	Bituminous premolded	40	60.0	37.5	2.5	90.0	10.0	0.0	...	....	17.5	82.5	13.7	0.0	86.3	...
Totals and Averages		368	55.2	42.4	2.4	90.5	8.6	0.9	...	....	....	....	....	...	....	...
2	J-1	201	57.0	36.0	7.0	86.0	10.0	4.0	1.5	5.5	8.5	83.0	6.5	6.5	72.0	...
	J-4	77	59.7	39.0	1.3	93.5	6.5	0.0	1.3	27.3	15.6	71.4	3.9	7.8	62.3	...
Totals and Averages		278	57.7	36.8	5.5	88.1	9.0	2.9	...	....	....	....	....	...	....	...
3	J-1	166	34.8	50.6	14.6	67.6	24.6	7.8	0.6	16.8	26.2	72.6	15.6	2.7	66.6	0.0
	J-4	70	48.6	44.3	7.1	75.7	20.0	4.3	...	20.0	14.3	85.7	....	...	84.3	...
Totals and Averages		236	39.0	48.7	12.3	70.0	23.2	6.8	...	....	....	....	....	...	....	...
4	J-1	195	11.3	76.9	11.8	69.2	26.7	4.1	0.5	16.4	27.7	66.2	22.1	4.1	52.8	...
	J-2	100	35.0	61.0	4.0	82.0	16.0	2.0	...	19.0	17.0	75.0	13.0	3.0	63.0	...
	J-4	40	40.0	55.0	5.0	90.0	10.0	0.0	...	12.5	2.5	85.0	5.0	2.5	97.5	...
	Fiber	182	46.2	48.9	4.9	80.8	18.7	0.5	...	....	15.9	84.1	18.7	6.0	75.3	...
Totals and Averages		517	30.4	62.3	7.3	77.4	20.5	2.1	...	....	....	....	....	...	....	...
5	J-2	128	42.2	47.6	10.2	67.1	22.7	10.2	...	31.3	10.2	85.9	14.8	2.3	70.3	...
6	Bituminous premolded	10	10.0	90.0	0.0	30.0	50.0	20.0	...	....	0.0	100.0	0.0	0.0	100.0	...
4 to 16	4-in. open	237	27.7	62.1	10.2	65.4	24.9	9.7	...	....	10.1	65.4	7.2	8.4	31.2	...

[1] Four-year-old fiber joints surveyed during February and March, 1943.

TABLE 39
SUMMARY OF DATA RELATING TO SPALLING AT CONTRACTION JOINTS
(Survey made during winter of 1939-40)

Age years	Type of Joint	Number of Joints	Number of Joints, percentage													
			With spalling										With joint edged			With concrete broken by asphalt
			Along joint			At center junction			Caused by dirt in joint		Affected by edging					
			None	Slight	Bad	None	Slight	Bad	Yes	No	Yes	No	High	Low	Level	
2	J-1	209	65.0	33.5	1.5	89.0	10.0	1.0	3.8	87.1	7.2	84.7	8.3	5.1	72.2	...
	J-4	77	72.7	24.7	2.6	89.6	9.1	1.3	...	87.0	1.3	85.7	7.8	7.8	71.4	...
Totals and Averages		286	67.1	31.1	1.8	89.2	9.5	1.1	...	....	...	....	...	...	....	...
3	J-1	166	63.3	32.5	4.2	88.0	8.4	3.6	5.4	74.1	13.3	36.7	14.5	2.4	27.1	0.6
	J-4	70	78.6	21.4	0.0	98.6	0.0	1.4	4.3	67.1	2.8	61.4	2.8	...	48.6	...
Totals and Averages		236	67.8	29.2	3.0	91.1	5.9	2.9	...	....	...	....	...	...	....	...
4	J-1	195	66.7	27.7	5.6	87.6	11.3	1.1	9.2	65.1	1.2	22.1	1.2	...	27.7	...
	J-2	100	66.0	29.0	5.0	86.0	12.0	2.0	3.0	57.0	1.0	18.0	2.0	1.0	21.0	...
	J-4	40	80.0	17.5	2.5	95.0	5.0	0.0	5.0	72.5	...	5.0	...	...	25.0	...
Totals and Averages		335	68.1	26.9	5.1	88.0	10.8	1.2	...	....	...	....	...	...	....	...
5	J-2	128	46.1	46.1	7.8	71.1	18.0	10.9	5.5	88.3	9.4	61.7	9.4	3.1	64.1	...

The data in Table 38 indicate that spalling along expansion joints and at the junction of the transverse and longitudinal joints increased with age. Spalling along expansion joints occurred at 44.8 per cent of the one-year-old joints, 42.3 per cent of the two-year-old joints, 61 per cent of the three-year-old joints, 69.6 per cent of the four-year-old joints, 57.8 per cent of the five-year-old joints, 90 per cent of the six-year-old joints, and 72.3 per cent of the 4-in. open joints, ranging in age from four to 16 years. Spalling at the junction of expansion and longitudinal joints occurred at 9.5 per cent of the one-year-old joints, 11.9 per cent of those two years old, 30 per cent of those three years old, 22.6 per cent of those four years old, 32.9 per cent of those five years old, 70 per cent of those six years old, and 34.6 per cent of the 4-in. open joints. While there was no uniform increase in spalling with age, the older pavements in some cases showing less spalling than those built later, there appears to be a general trend toward an increase with age. The data also show that spalling along the joint was much more prevalent than at the junction of transverse and longitudinal joints.

Available comparisons indicate that somewhat more spalling occurred at J-1 expansion joints than at the other types of air-chamber joints. Since this joint has no apparent features which would affect spalling any more than other types, this observation is difficult to explain. The pavements considered in the comparisons all carried approximately the same average daily traffic. One difference observed by engineers who made the survey was that, for some unknown reason, a greater portion of the J-1 expansion joints were finished with high edges, a condition which reasonably could lead to spalling. It is also possible that the wider flange on the seal of this joint caused more honeycomb and thus resulted in greater spalling.

From the limited data available, it would appear that slightly less spalling occurred along fiber expansion joints than adjacent to air-chamber expansion joints. At four years, the only age at which a direct comparison can be made, spalling occurred at 89 per cent of the J-1 expansion joints, 65 per cent of the J-2 expansion joints, 60 per cent of the J-4 expansion joints, and 54 per cent of the fiber joints. Assuming these data to be truly representative, it is probable that the greater amount of spalling along air-chamber joints is due to the effect of their metal seals in contributing to honeycomb adjacent to the joint and in weakening the edges of the concrete along the joints. There was no marked difference between air-chamber joints and fiber joints, as far as spalling at the junction between transverse and longitudinal joints was concerned.

Spalling along one-year-old fiber joints was slightly less than along bituminous premolded joints of the same age, but there was little difference between them in the case of spalling at the junction between transverse and longitudinal joints. Ninety per cent of the six-year-old bituminous premolded joints had slight spalling, which is considerably higher than for any of the other types; however, none of these joints was classified as having bad spalling.

About 72 per cent of the 4-in. open joints showed spalling of the edges adjacent to the joints, 62 per cent being classified as slight and 10 per cent as bad spalling. Spalling at the junction of transverse joints with the center joint was about as prevalent as in the case of other types of joints.

The data given in Table 39 for metal contraction joints are in general agreement with those for air-chamber expansion joints, although the tendency for spalling to increase with age is not so apparent. Spalling occurred at a greater proportion of J-1 contraction joints than at other contraction joints, but, as in the case of expansion joints, more of the J-1 joints were edged high and, in the opinion of those who made the examinations, this was responsible for the greater spalling.

(c) Condition of Bituminous Caps and Asphalt Filler

The purpose of the bituminous premolded caps installed on some of the air-chamber expansion joints, particularly the J-1 joint, and the asphalt filler used on both air-chamber and premolded joints, was to protect the copper seal, when used, and to prevent dirt and other foreign material from entering the joints from the top. The observations made during the several field investigations all showed conclusively that the premolded caps had a very short serviceable life, and that both the caps and the poured asphalt fillers became ineffective in performing their functions within a short time after installation. The premolded caps cracked and split under traffic and when the joints closed. In some cases, pieces of the cap were pulled out of the joint by traffic and the seal was exposed. Practically all joints examined had dirt and other foreign material under the cap. The poured asphalt fillers were equally as ineffective as the premolded caps. While they did not split and crack as the caps did, they did not prevent foreign material and water from entering the joint. Some of the investigators thought that the poured filler offered slightly more protection for the copper seal, but that the difference was so small as to be of no practical significance.

A summary of the results of the 1939 and 1943 investigations relating to the caps and fillers is given in Table 40. Premolded caps were installed almost universally on the J-1 joints included in these investigations. Only a few of the J-2 joints were installed with premolded caps, a metal installing bar being used and the seal covered with a poured filler. Premolded caps were used extensively on early J-4 joints, but their use was discontinued in favor of poured fillers the last year metal joints were used.

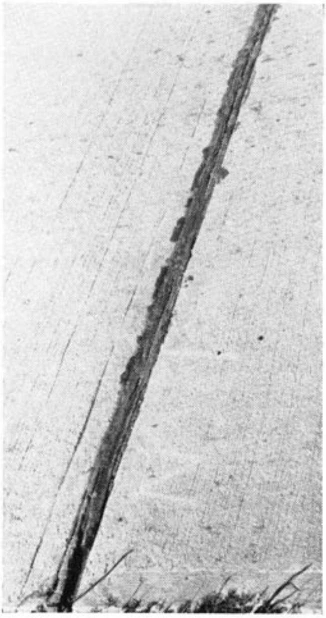


FIG. 82. TYPICAL CONDITION OF PREMOLDED CAP ON J-1 EXPANSION JOINT

Considering the two-year-old J-1 expansion joints, of which 90 per cent were installed with premolded caps, 87.5 per cent of these joints still had caps, which represents a loss in two years of 3 per cent. Of the caps still in place, 53.4 per cent were broken, split or disintegrated, and of little value as far as protecting the seal was concerned. Figure 82 shows a condition which may be considered typical of the premolded caps on J-1 expansion joints.

Approximately 96 per cent of the three-year-old J-1 expansion joints were installed with premolded caps, of which about 2 per cent were missing when the joints were examined. Eighty-six per cent of the three-year-old J-4 expansion joints were installed with premolded caps, of which 33 per cent were missing. Approximately 68 per cent

TABLE 40

SUMMARY OF DATA RELATING TO CONDITION OF PREMOLDED CAPS AND FILLER MATERIAL ON EXPANSION JOINTS

(Survey made during winter of 1939-40)[1]

Age years	Type of Joint	Number of Joints	Number of Joints, percentage											
			Expansion									Contraction		
			Installed with cap		With cap in place		With broken, split and disintegrated caps[2]	Covered with asphalt		Cap and/or filler effective in keeping out dirt and water		Cap and/or filler effective in keeping out dirt and water		Not poured with asphalt
			Yes	No	Yes	No		Yes	No	Yes	No	Yes	No	
1	Fiber	328	....	....	....	....	....	69.5	....	11.6	88.4	....	....	...
	Bituminous premolded	40	....	....	....	....	....	100.0	....	15.0	85.0	....	....	...
2	J-1	201	90.0	10.0	87.5	2.5	53.4	84.5	3.0	17.5	82.5	14.8	60.8	8.6
	J-4	77	0.0	100.0	0.0	0.0	....	0.0	0.0	16.9	83.1	16.9	68.8	...
3	J-1	166	95.8	4.2	93.9	1.9	68.1	92.8	1.1	1.3	98.7	14.5	44.5	3.6
	J-4	70	85.7	14.3	57.2	28.5	85.0	55.7	1.5	1.4	98.6	41.4	41.4	2.8
4	J-1	195	100.0	0.0	90.8	9.2	71.2	79.0	11.8	2.1	97.9	14.4	62.1	...
	J-2	100	10.0	90.0	10.0	0.0	80.0	10.0	0.0	0.0	100.0	25.0	35.0	2.0
	J-4	40	100.0	0.0	82.5	17.5	57.6	60.0	22.5	2.5	97.5	30.0	32.5	...
	Fiber	182	....	....	....	....	....	56.0	....	2.7	97.3	....	....	...
5	J-2	128	7.8	92.2	6.2	1.6	100.0	7.8	0.0	4.7	95.3	6.3	90.6	7.0
6	Bituminous premolded	.....	....	....	....	....	....	100.0	....	10.0	90.0	....	....	...
4-16	4-in. open	237	....	....	....	....	....	53.2	....	4.8	95.2	....	....	...

[1] Four-year-old fiber joints surveyed during February and March, 1943.
[2] Percentage based on number of caps now in place.

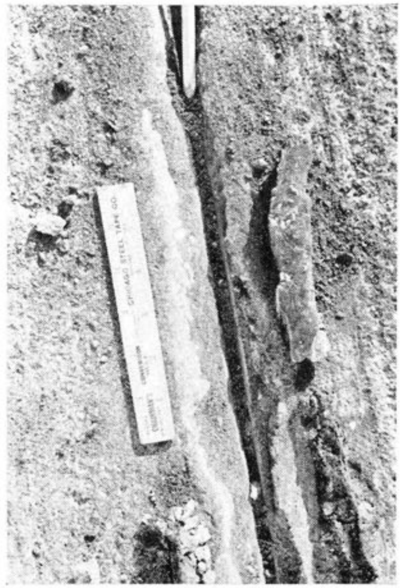
of the J-1 caps and 85 per cent of the J-4 caps on the three-year-old joints were split, broken, or disintegrated.

All of the four-year-old J-1 and J-4 expansion joints and 10 per cent of the J-2 expansion joints were installed with premolded caps. The table shows losses of 9.2, 0.0, and 17.5 per cent for J-1, J-2, and J-4 joints, respectively. Of those which remained in place, approximately 71, 80, and 58 per cent of the caps on the J-1, J-2, and J-4 joints, respectively, were split, broken, or disintegrated.

The only five-year-old metal joints examined in 1939 were the J-2. Only 8 per cent of these were installed with premolded caps and approximately 20 per cent of the caps had disappeared; all of the remaining caps were split, broken, or disintegrated.

All the joints had been fairly well maintained. The close agreement between the percentage of joints installed with premolded caps and the percentage of joints with caps covered with asphalt filler, indicates that most of the joints had been poured at least once. In spite of this, the data show that the caps and fillers on expansion joints definitely were not effective in keeping out foreign material. This was true of approximately 85 per cent of the one- and two-year-old joints and of practically all of the joints more than two years old. The survey indicated that the poured filler on contraction joints was somewhat more effective in this respect than the cap or filler on expansion joints. However, this difference may be due, at least in part, to the fact that many of the contraction joints were so tightly closed that it was impossible to determine whether foreign material had entered.

Figures 83 to 86, inclusive, are typical examples of the failure of caps and fillers to keep foreign material out of the several types of joints. Figure 83 shows a J-1 expansion joint with cap and filler removed, exposing a thick deposit of dirt and sand on top of the copper seal. The removed cap lying on the pavement shows how the caps become broken and split. A J-2 contraction joint with the filler removed to expose a thick layer of dirt on the copper seal is shown in Fig. 84. Figure 85 is a closeup of a fiber expansion joint with the bituminous filler removed, showing a deposit of dirt above the fiber material and the fiber pulled away from the concrete, leaving a space which eventually will become filled with foreign material. A closeup of a 4-in. open joint, showing the general condition of the filler in this type of joint, is given in Fig. 86. The photographs show conclusively that none of the caps or fillers was effective in keeping foreign material out of the various types of joints covered by the investigation.



Figs. 83 and 84 (Above) and 85 (Below, at Left). Typical Examples of Foreign Material — Respectively under Cap and Filler on J-1 Expansion Joint, in Top of J-2 Contraction Joint, under Filler on Fiber Expansion Joint

Fig. 86. Typical Condition of Filler in 4-in. Open Joint

(d) Infiltration of Dirt and Water into Joints and Transverse Cracks

One of the primary purposes of pavement joints is to provide sufficient expansion space to prevent blowups. The various investigations show that the joints commonly used fulfill this requirement in part only; in other words, the most they may do is postpone for a number of years the occurrence of blowups. The investigation of the 4-in. open joints in 1932 revealed that these joints closed at an average rate of 1 in. per year due to the infiltration of solid material into the transverse cracks between joints. The several investigations of air-chamber joints showed that, in spite of the close spacing of the joints, transverse cracks occurred and the pavements were subject to growth due to infiltration into these cracks. Added to this was the infiltration into the joints themselves which reduced the available expansion space. This was true of all types of joints, including those with copper seals. Observations proved that even when the seals had not split and permitted infiltration from the top, the expansion space had been reduced by soil working up into the joint from the subgrade. Even experimental joints, with seals on the bottom as well as the top and ends, did not prevent infiltration.

In the 1939 investigation, a study was made to determine the extent to which joints were being affected by infiltration. It was noted whether water had been in the joint, what the width of the joint opening was, how much dirt was in the joint, and the temperature at the time of examination. These examinations were confined to the expansion joints because it was almost impossible to examine the small space between slabs at contraction joints. The results are given in Table 41.

From 65.6 to 91.3 per cent of the air-chamber expansion joints had water in the joint opening or showed evidence of water having been there, chiefly the latter, because the survey was made during a period of very dry weather. A large percentage of joints, ranging from 55 to 90 per cent, had dirt in the opening. Many of the premolded joints contained dirt, even those only one year old, the dirt having entered during the first winter when the concrete contracted and pulled away from the joint material. Age appeared to have little effect in this respect on premolded joints; almost as large a proportion of the one-year-old joints contained dirt as those six years old.

The effect of age on the accumulation of dirt in air-chamber joints is shown by the average maximum height of dirt. It was found that dirt in the joint usually was highest at the ends, tapering off toward the center; in general, very little dirt was found at the center of the

TABLE 41
SUMMARY OF DATA RELATING TO CONDITION OF INTERIOR OF EXPANSION JOINTS
(Survey made during winter of 1939-40)[1]

Age years	Type of Joint	Number of Joints Examined	Joints in Which There Had Been Water per-centage	Dirt in Joints		Joint Width, in.		
				Per-centage of total	Average maxi-mum height in.	When in-stalled	When ex-amined[2]	Dif-ference[3]
1	Fiber	328	....	88.0	....	0.75	0.88	0.13
	Bituminous premolded	40	....	85.0	....	1.25	1.19	−0.06
2	J-1	201	74.1	55.0	0.97	1.00	0.79	−0.21
	J-4	77	66.2	70.0	1.34	0.75	0.63	−0.12
3	J-1	166	66.2	67.0	2.06	0.75	0.46	−0.29
	J-4	70	87.1	83.0	1.95	0.75	0.59	−0.16
4	J-1	195	65.6	64.0	2.46	0.75	0.25	−0.50
	J-2	100	75.0	82.0	1.95	0.81	0.46	−0.35
	J-4	40	91.3	87.0	2.88	0.75	0.31	−0.44
	Fiber	182	....	68.0	....	0.75	0.77	0.02
5	J-2	128	76.6	77.0	2.71	0.81	0.42	−0.39
6	Bituminous premolded	10	....	90.0	....	0.50	0.53	0.03
4 to 16	4-in. open	227	....	57.0	3.74	4.00	1.63	−2.37

[1] Four-year-old fiber joints surveyed during February and March, 1943.
[2] Average temperature at time of examination was approximately 38 deg. F. lower than that at time of installation, except 4-year-old fiber joints, where difference was approximately 27 deg. F.
[3] A minus value indicates that the joint opening was less at the time of examination; a plus value that it was greater.

joints. The values given in Table 41, averages of the maximum heights for each particular type and age of joint, were determined by dividing the total maximum height for any one classification by the number of joints in that group containing dirt. For the air-chamber expansion joints, the average height of dirt ranged from 0.97 in. to 2.89 in., showing a definite increase with age. The dirt had accumulated between the metal side walls of the joint, having entered from the ends and top. Dirt was also found between the side walls of the joint and the concrete, a condition also true in the case of premolded joints. The typical condition of air-chamber joints containing dirt is shown by Figs. 87 to 89, inclusive. These are photographs of J-1, J-2, and J-4 expansion joints, respectively, showing that all are subject to infiltration. This fact also is apparent from Table 41, which shows that a majority of each type contained dirt and that the average maximum height was approximately the same for each.

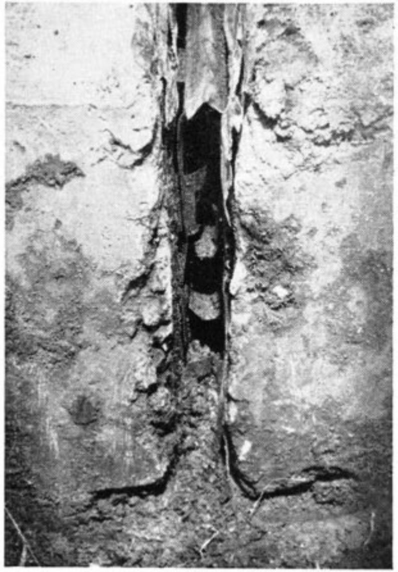


FIG. 87
TYPICAL EXAMPLE OF DIRT IN J-1 EXPANSION JOINT

FIG. 88
TYPICAL EXAMPLE OF DIRT IN J-2 EXPANSION JOINT

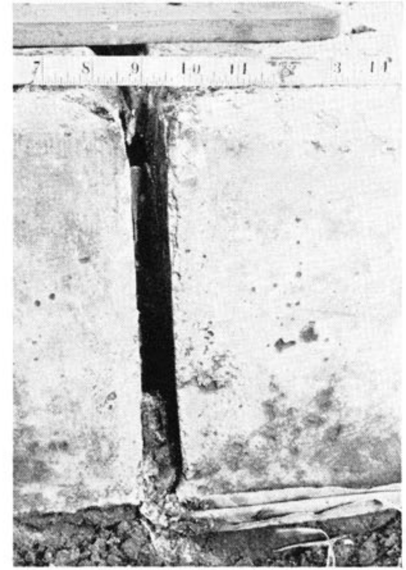


FIG. 89
TYPICAL EXAMPLE OF DIRT IN J-4 EXPANSION JOINT

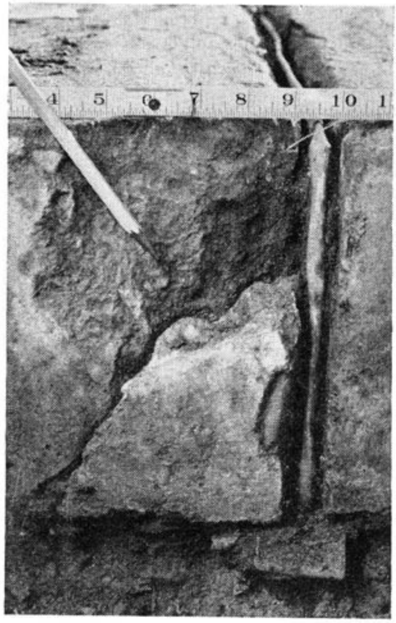


FIG. 90. TYPICAL EXAMPLE OF FRACTURE IN EDGE OF PAVEMENT DUE TO FOREIGN MATERIAL IN END OF JOINT

Accumulations of dirt in joints decrease the available expansion space and consequently reduce the length of time the joint will be effective in preventing blowups. While no blowups were found in 1939 in pavements with air-chamber joints, conditions existing then indicated that it will be only a matter of a few years until such failures begin to occur. Of more immediate importance are the failures at the extreme corners of the concrete slabs adjacent to the ends of the joints, as shown in Fig. 90. Failures of this kind usually result from accumulations of dirt in the ends of the joint. As the concrete expands, the concentrated forces due to the dirt in the ends of the joint produce stresses too great for the concrete to resist, and the corner breaks off. Such failures are by no means uncommon, being found in 2 to 6 per cent of the contraction joints and in 5 to 17 per cent of the air-chamber expansion joints covered by the 1939 investigation.

As previously mentioned, dirt works up into the joint from the subgrade. This was shown definitely by a condition found at many of the 4-in. open joints. Solid deposits of dirt, sometimes extending to within an inch of the surface of the pavement, were found in the bottom of these joints. The dirt lay in the bottom of the joint opening and was covered with a layer of asphalt, the average maximum height of the dirt being almost 4 in.

In maintaining these joints, it is common practice in hot weather to trim off the excess asphalt which extrudes as the joint closes and the asphalt swells. Unquestionably dirt working up into the joint due to pumping or some other cause, and entering from the top when the filler was pulled away from the concrete, progressively displaced the asphalt filler until after a number of trimmings very little asphalt was left in the joint. This condition in the 4-in. joint, together with the fact that dirt also worked up into metal and premolded joints, indicates that it is useless to spend money for a permanent seal on the top and ends of the joint, if such a seal were available, unless adequate measures are taken to keep the soil from entering the joint from the subgrade. It also shows that the effectiveness of an open joint can be improved by periodic cleaning to its full depth and repouring.

It has been pointed out that joints close permanently because of growth due to infiltration of solid material into transverse cracks. Table 41 shows the extent to which this had occurred in the joints covered by the 1939 and 1943 investigations. Before proceeding with the discussion, it should be pointed out that the data on reduction of expansion space are only approximate, because of the difficulties encountered in measuring the expansion space and the fact that no information as to the expansion space or width at the time of installation is available other than the nominal width of the joint. The data, however, are sufficiently accurate to show general trends.

The average air temperature at the time the joints were examined in 1939-1940 was approximately 38 deg. F. lower than at the time the joints were installed. Hence, if nothing else occurred to change the length of the pavement, the joint openings at the time of examination should have been wider than at the time of installation. Assuming that the change in average temperature of the concrete was the same as the difference in air temperature (38 deg. F.) and that there were no intermediate cracks in the 30-ft. panels between air-chamber joints, each joint should have opened 0.076 in., irrespective of any initial or subsequent shrinkage due to loss of moisture and restraint due to subgrade resistance. Tests[10] indicate the subgrade resistance does not affect appreciably the free movement of slab panels up to 75 ft. in

[10] Cashell, H. D., and Benham, S. W., "Experiments with Continuous Reinforcement in Concrete Pavement — A Five-Year History," Proc. Highway Research Board, pp. 37-38, Vol. 23, 1943.

length. Similar conclusions were drawn from unpublished tests made by the Illinois Division of Highways. In this connection, it is pointed out that the examinations were made during a period of extremely dry weather when shortening due to moisture loss probably was at its maximum. Table 41 shows that, with the exception of the one-year-old pavements with fiber joints and the six-year-old pavements with bituminous premolded joints, all the joints had partially closed from their initial widths. The one-year-old pavements with fiber joints had very few transverse cracks and these were held tightly together by the wire mesh reinforcement used in those pavements. Thus the length of the pavement slabs between joints was not affected by infiltration of dirt into cracks, and change in joint opening had to be due to changes in temperature and moisture. With a 50-ft. joint interval and a temperature change of 38 deg. F., the theoretical average increase in joint opening would be 0.125 in., as compared to an actual increase of 0.13 in., shown by Table 41. Such close agreement is probably a coincidence, but the data show that if cracks do not form or are prevented from opening, pavements will expand and contract in accordance with the theory of thermal expansion.

In the supplementary investigations made in 1943 and 1945 of pavements with fiber joints, special attention was given to the effectiveness of the wire mesh reinforcement in these pavements in holding transverse cracks closed. Almost every district reported that the reinforcement was effective in holding transverse cracks tightly closed, some adding that cracks were visible only on careful examination. One district, where yearly crack surveys had been made, cited a case where cracks observed in 1941 were so tightly closed that the maintenance patrolmen had overlooked them one or more times when filling cracks with asphalt. In 1943, cracks which had opened up were found on only three of the 18 sections included in the survey. One section, which had a total of 217 transverse cracks, had only one open crack, its opening being ¼ in. On another section, with 187 cracks, one crack had opened ⅛ in. On the other section, out of 346 cracks only 30 were opened, the amount of opening varying from ⅛ to ½ in. The open cracks in the latter section were not distributed over the entire section, but occurred only on one part of the section and mainly on fills, indicating that they probably were due to some condition peculiar to that part of the section.

Perhaps the best evidence of the effectiveness of wire mesh reinforcement in holding slabs together after transverse cracks occur is shown by the width of the fiber joints and air-chamber joints in four-year-old pavements. The pavements containing fiber joints were reinforced with wire mesh; those with air-chamber joints contained no reinforcement. Table 41 shows that while the air-chamber joints had closed an average of from 0.35 in. to 0.50 in., the average width of the fiber joints was practically the same as when they were installed.

No logical reason can be given for the openings at bituminous premolded joints in the six-year-old pavement being wider than when installed. This pavement was badly cracked, and there was plenty of opportunity for solid matter to enter the cracks and cause the joints to close. However, only 10 joints on one section of pavement were examined and they may not have been representative of average conditions.

In all the other types of joints, the available expansion space progressively decreased with the age of the pavement. The older joints were closed proportionately more than the newer joints, the approximate average rate of closure of air-chamber expansion joints being 0.10 in. per year. If the joints continue to close at this rate, the original expansion space of 0.75 in. provided at each expansion joint will have been taken up after approximately eight years of life.

The available expansion space also is reduced by dirt entering the joints themselves, and this fact must be considered in estimating the period of time through which the joint will be effective in relieving stresses due to expansion of the concrete. Upon the basis of present knowledge, it appears that blowups will begin to occur in a very few years in some of the older pavements built with joints at close intervals, unless something is done to furnish further relief.

It is interesting to note that the average width of the 4-in. open joints was only 1.63 in., in spite of the relief afforded by recutting many of the original joints and cutting new ones. This represents an average net closure of 2.37 in. The average age of the pavements containing 4-in. joints being about eight years, the rate of closing was almost 0.3 in. per year, disregarding the additional relief mentioned above.

It may be concluded that joints in unreinforced pavements, such as those covered by this bulletin, afford only temporary relief from the forces that cause blowups, and at best serve only to delay the conditions they are intended to prevent. The seals on the air-chamber expansion joints have failed to prevent infiltration into the expansion space and, even had they proved satisfactory in that respect, they would be of little practical value as long as foreign material entering transverse cracks robs a joint progressively of its available expansion space. It appears from the available data that the use of wire mesh reinforcement to hold transverse cracks closed so that infiltration will not occur, in conjunction with one of several kinds of premolded joints or wood joints, spaced judiciously, may be the most practical and economical solution to this problem.

(e) Condition of Premolded Joint Fillers

Experience with the use of premolded joints in highways constructed or supervised by the Division of Highways was quite limited until after 1938, when this type of joint was adopted as a standard. A few installations had been made prior to that time, but these were not sufficiently comprehensive to justify conclusions. Hence, no studies were made of this type during any of the field investigations conducted by the Division of Highways until 1939, after a considerable mileage of fiber joints had been installed. On the other hand, the University committee, in its investigation in 1937, examined a great many joints of this kind, both in Illinois cities, where they had been used for many years, and in other states throughout the country. The committee found that the premolded joints possessed certain defects, but from wide observations they concluded that results superior to those which had been secured with air-chamber joints in Illinois could be secured more economically with premolded joints, in spite of defects which were found in such joints. It was largely on the strength of their recommendations that the use of premolded joint fillers was adopted in 1938.

The majority of the premolded joints included in the 1939 investigation had not been in service long enough to warrant definite conclusions regarding the durability of the material and its ability to withstand the destructive forces of weather and traffic. Only 10 joints more than one year old were examined, these being bituminous premolded joints in a section of pavement six years old. The remainder of the joints of this type, including 328 fiber joints in 33 sections of pavement and 40 bituminous premolded joints in four sections of pavement, were one year old. The examinations, however, revealed some defects in the materials which unquestionably will have a decided effect on the service life of these joints.

The fiber joints one year old were little affected as far as disintegration, slumping, and extrusion of the joint material were concerned. About 3 per cent of the joints showed signs of disintegration; the material had slumped in 11 per cent; only one joint was reported to have extruded.

At four years, about 20 per cent of these joints showed disintegration, only one joint was reported to have slumped in the opening, and 11 per cent had extruded. One district reported that the joint material had disintegrated badly at the ends of the joints. With regard to slumping and extrusion, it was observed that the fiber material in the four-year-old pavement had risen. These joint fillers were installed ½ in. below the surface of the pavement, and when examined in 1943 many of them were flush with the top of the pavement. This may have been due to water freezing below the joint and gradually lifting it.

The fiber material apparently is more susceptible to weathering than the bituminous premolded material; at least the effects become apparent earlier. About 33 per cent of the one-year-old fiber joints and 40 per cent of those four years old were split and pulled apart to such an extent that water and dirt could enter, as compared with 2.5 per cent of the bituminous premolded joints one year old reported in that condition. The material in 14 per cent of the one-year-old fiber joints and 63 per cent of those four years old had become very soft; samples submitted from the field in sealed containers definitely had less resistance to compression than unweathered material. None of the bituminous premolded joints was reported as in this condition. This tendency of fiber joint material to soften has been detected in the laboratory in making freezing and thawing, wetting and drying, and absorption tests. The material becomes soft and lifeless when wet. It is probable that a much higher percentage of the fiber joints would have been found in this condition had they been examined during wet weather. Such a condition is undesirable and undoubtedly affects the service life of the joint, because it enables solid material to work up into the joint from the subgrade or to be driven into the joint material by the action of traffic.

An example of how soil works up into a joint is shown in Fig. 91. This joint, of the bituminous premolded type, was six years old at the time the photograph was taken. The material had disintegrated and the subgrade had risen and replaced approximately 3 in. of the joint material.

In the case of the bituminous premolded joints, 20 per cent of those one year old had slumped in the joint opening and 25 per cent of them were extruded. The limited number of joints of this type which were six years old when examined indicate that splitting, softening, and disintegration are a function of age; however, the data are by no means conclusive.

The data from the several investigations are not of sufficient scope

FIG. 91. PHOTOGRAPH SHOWING HOW SOIL FROM SUBGRADE WORKS UP INTO PREMOLDED JOINT

to establish the serviceable life of premolded fiber or bituminous premolded joint fillers. They show that these materials possess certain defects which are bound to reduce their serviceable lives, but, in spite of these, this type of joint can be expected to perform better and more economically than air-chamber joints. This conclusion is drawn chiefly from the results which have been obtained with premolded joints in many Illinois cities and in other states. It appears that until a joint is developed which possesses many more of the desirable requirements than do any of the types now in use, the premolded fillers are the most practical and economical.

Experience in Illinois is confined almost entirely to fiber joints and bituminous premolded joints. Other types were examined by members of the University committee traveling in other states, who found that joints made from wood boards show a great deal of promise. They are easy to install, hold their alignment, furnish good support to load transmission devices, and appear to be durable. They are more resistant to compression than other types, and hence set up greater compressive forces in the concrete than other types of fillers. Many engineers are of the opinion that this is an advantage, believing that compression will offset some of the tensile stresses induced by wheel loads, and that as long as the compressive stresses do not reach critical limits, they are beneficial. Since wood joints appear to have some desirable advantages, it is believed that their properties should be given further study. Some progress already has been made in this direction with the installation of a number of wood joints in the Armington Experimental Road, which is discussed elsewhere in this bulletin (pages 197–243).

(f) Condition of Load Transmission Devices

It is impossible to make a detailed examination of load transmission devices without destroying the surrounding pavement. For that reason, the examinations made of these joint accessories were cursory in all the investigations. In one of the investigations which included approximately 183, 176, and 140 miles of pavement containing conventional dowels, L-1, and L-2 load transmission devices, respectively, special attention was given to the relation between load transmission devices and corner breaks, transverse cracks near the joints, and differential settlement of the ends of slabs adjacent to joints, with the hope that such failures might reveal failures of the load transmission devices. It was also thought that bent, corroded, and misaligned devices could be detected when examining the joints, especially in the case of metal joints in which the devices were partially exposed when the copper seals were removed.

Of these defects, only corner breaks showed any indication of being influenced by the type of load transmission device. Pavements with conventional dowels had 18 exterior and 23 interior corner breaks per hundred miles. Corresponding values for pavements with L-1 devices were three exterior and three interior breaks per hundred miles. The pavements with the L-2 device had 31 exterior and 14 interior corner breaks per hundred miles. While corner breaks did not occur with sufficient frequency to be of serious consequence, nevertheless these data are distinctly favorable to the wing anchor device. It is interesting to note that the pavements with L-2 angles had considerably more exterior breaks than pavements with other devices. The reason for this may be in the design of the device itself. The arrangement of these devices, alternate angles extending under opposite slabs, places the center of the first point of support for one corner at each edge of the pavement, 18 in. from the edge. It is conceivable that such an arrangement would be more conducive to corner breaks than where companion corners have mutual support 6 in. from the pavement edge, as is the case when wing anchor devices and conventional dowels are used.

Crushing of the concrete around ¾-in. conventional dowel bars, a condition commonly known as funneling because of the conical shape of the failure, was found by one district at a limited number of dowels. Had this type of failure been frequent, it would suggest a definite weakness in the ¾-in. conventional dowel. However, in comparison to the number of dowels examined during the investigation, the number of failures was almost negligible and the data do not justify definite conclusions with respect to funneling.

(g) Pavement Roughness Induced by Joints

The public judges a highway by the smoothness of its surface, as reflected by the riding comfort, or lack of it, that the pavement provides. A great deal of effort is expended in securing a high degree of initial smoothness, and it is equally important that this smoothness be retained to the greatest extent possible.

It has been found that any break in the continuity of a concrete pavement, whether a natural crack or a transverse joint, may affect the smoothness of the pavement. Some of the reasons advanced by authorities are swelling of some types of soil immediately adjacent to the crack or joint due to surface water entering the opening, freezing of water in the subgrade, differential settlement of the edges of adjacent slabs, and warping of the slab due to temperature and moisture differentials. Surface irregularities also may occur at joints during the pavement construction, because the finishing operation must be interrupted while the screeds are raised over the joint, a practice which may leave a high spot at the joint. Whatever the cause, the result is the same, a poorer riding pavement.

Surface roughness at joints will affect riding quality to varying degrees. In one instance, the passenger may be aware of roughness only by the rhythmic thud of the tires passing over the closely and equally spaced joints in rapid succession, and may feel no physical discomfort. In another instance, the difference in elevation may be of sufficient magnitude to impart a distinct vertical motion to the vehicle. Both may prove annoying and uncomfortable to a passenger, especially if continued for any length of time.

It would be extremely difficult to evaluate roughness on the basis of the psychological and physical reactions of passengers, even if these factors could be measured, for they vary with individuals as well as with the joint spacing, the design of the motor vehicle, its mechanical condition, its age, and the speed at which it is driven. The relative roughness of a pavement can be determined, however, by comparing the measured contour of its surface with a standard of smoothness which experience has shown to afford satisfactory riding qualities. This was the method used to study the roughness of the pavement adjacent to the joints surveyed in 1939. During that investigation, elevations were measured at some 2,500 joints, readings being taken at one-ft. intervals along each of the four normal wheel paths, beginning 5 ft. back of the joint and extending 5 ft. beyond; also directly over the filler and at points on the concrete surface immediately adjacent to the filler. In 1943, elevations were again measured at 182 fiber joints on 18 sections, two in each of nine districts, included in the 1939 survey.

The standard used as a basis of comparison is the specification for surface smoothness under which the pavement was built. Most of the pavements included were built to comply with the requirement that variations in the pavement surface should not exceed ⅛ in. in 10 ft. The only exceptions were the pavements built prior to March 23, 1931, in which 4-in. open joints were installed. The specifications in effect before that date required that the variations should not exceed ¼ in. in 10 ft.

Elevation of the ends of the slabs adjacent to joints has already been mentioned as a cause of pavement roughness. Another cause is the extrusion of the cap or filler, or the joint material itself, when the joint is of the premolded or poured type. In relatively wide joints, such as the 4-in. open type, insufficient filler may be responsible for roughness, because the wheels in passing over the depressed filler change vertical position rapidly, and this motion is transferred in some degree to other parts of the vehicle. The data from the investigations are analyzed with respect to these features.

(1) Effect of Filler on Roughness. Tables 42 to 46, inclusive, give the data relative to the height of fillers. A summary of all the data for expansion joints, Table 42 gives the average height of the filler with respect to the adjacent concrete surfaces for each type and age of joint. Table 43 contains the same information for contraction joints. The number and percentage of wheel paths over expansion joints having high fillers, and the maximum and average heights of high fillers for each type and age of joint whose original width was 1 in. or less, are given in Table 44. Data for contraction joints similar to those in Table 44 are given in Table 45. Data relating to high and low fillers, for the 2-in. bituminous premolded joint and the 4-in. open joint, are given in Table 46.

The fillers on both expansion and contraction joints were generally lower than the adjacent concrete surfaces (Tables 42 and 43), and no wide variations existed except in the case of the 4-in. open joints. The average level of the filler below the concrete for all except the 4-in. joints was 1/32 in. to 1/16 in.,

TABLE 42
SUMMARY OF MEASUREMENTS TAKEN TO DETERMINE THE HEIGHT OF FILLER OVER EXPANSION JOINTS
(Survey made during winter of 1939-40)[1]

Age years	Type of Joint	Number of Joints	Average Height of Filler with Respect to Concrete Surface, in.				
			Wheel path				Average
			1	2	3	4	
1	Fiber Bituminous premolded	328 40	−0.041 −0.023	−0.061 −0.036	−0.059 −0.030	−0.040 −0.006	−0.050 −0.024
Totals and Averages		368	−0.039	−0.058	−0.053	−0.036	−0.047
2	J-1 J-4	201 77	−0.035 −0.052	−0.037 −0.077	−0.040 −0.088	−0.027 −0.075	−0.036 −0.079
Totals and Averages		278	−0.040	−0.048	−0.053	−0.040	−0.048
3	J-1 J-4	166 70	−0.027 −0.120	−0.028 −0.120	−0.032 −0.080	−0.026 −0.081	−0.029 −0.110
Totals and Averages		236	−0.055	−0.055	−0.046	−0.042	−0.053
4	J-1 J-2 J-4 Fiber	195 100 40 182	−0.008 −0.023 −0.017 −0.142	−0.003 −0.040 −0.025 −0.152	−0.002 −0.030 −0.026 −0.150	−0.007 −0.024 −0.053 −0.149	−0.006 −0.028 −0.030 −0.149
Totals and Averages		517	−0.059	−0.064	−0.061	−0.064	−0.062
5	J-2	128	−0.036	−0.042	−0.037	−0.028	−0.036
6	Bituminous premolded	10	−0.019	−0.078	−0.073	−0.116	−0.073
All Ages	4-in. open	242	−0.280	−0.270	−0.260	−0.280	−0.270

[1] Four-year-old fiber joints surveyed during February and March, 1943.

and there was no indication that the height of the filler is affected by the type or age of joint. It is obvious that such a small difference in elevation would not produce sufficient vertical displacement of the wheels of vehicles to be noticeable to passengers, especially on narrow joints such as those under discussion. On the other hand, at the temperature at which the examinations were made, it could be expected that the fillers on the 4-in. joints would be low, due to the relatively large movements caused by contraction of the concrete and to the concurrent shrinkage of the large volume of filler in the joint. The average depression at these joints was approximately ¼ in., which, because the joints were wide enough to permit wheels to roll down into the depression, probably would make them noticeable, if not disturbing, to passengers.

The data, however, do not give a complete picture of the relation of filler to riding quality, because they show nothing as to the riding qualities of individual joints; after all, it is the condition at each joint and each wheel path that determines whether or not a vehicle will ride smoothly over it. In

TABLE 43
SUMMARY OF MEASUREMENTS TAKEN TO DETERMINE THE HEIGHT OF FILLER OVER CONTRACTION JOINTS
(Survey made during winter of 1939-40)

Age years	Type of Joint	Number of Joints	Average Height of Filler with Respect to Concrete Surface, in.				
			Wheel path				Average
			1	2	3	4	
2	J-1 J-4	209 77	−0.090 −0.033	−0.090 −0.026	−0.086 −0.045	−0.079 −0.033	−0.085 −0.038
Totals and Averages		286	−0.075	−0.073	−0.075	−0.067	−0.072
3	J-1 J-4	166 70	−0.039 −0.016	−0.034 −0.017	−0.039 −0.026	−0.046 −0.022	−0.033 −0.022
Totals and Averages		236	−0.032	−0.029	−0.035	−0.039	−0.030
4	J-1 J-2 J-4	195 100 40	0.016 −0.008 0.017	0.017 −0.010 0.022	0.023 0.006 0.012	0.031 0.020 0.019	0.022 −0.004 0.018
Totals and Averages		335	0.009	0.010	0.017	0.026	0.014
5	J-2	128	−0.015	−0.016	−0.015	−0.011	−0.014

TABLE 44
SUMMARY SHOWING NUMBER OF WHEEL PATHS OVER EXPANSION JOINTS HAVING HIGH FILLERS AND THE MAXIMUM AND AVERAGE HEIGHTS OF THE FILLERS
(Survey made during winter of 1939-40)[1]

Age years	Type of Joint	Number of Joints Examined	Number of Wheel Paths Measured	Wheels Paths Having High Filler		Height of High Filler, in.	
				Number	Per-centage	Maximum	Average
1	Fiber Bituminous premolded	83 10	332 40	71 13	21.4 32.5	0.34 0.13	0.10 0.08
Totals and Averages		93	372	84	22.3	....	0.10
2	J-1 J-4	190 77	760 308	254 27	33.4 8.8	0.41 0.09	0.07 0.04
Totals and Averages		267	1,068	281	26.4	....	0.06
3	J-1 J-4	175 70	700 280	232 39	33.1 13.9	0.97 0.13	0.06 0.05
Totals and Averages		245	980	271	27.7	....	0.06
4	J-1 J-2 J-4 Fiber	195 100 40 182	780 400 160 728	351 116 49 52	45.0 29.0 30.6 7.1	0.25 0.22 0.25 0.15	0.08 0.05 0.08 0.04
Totals and Averages		517	2,076	568	27.4	....	0.06
5	J-2	118	472	137	29.0	0.19	0.06

[1] Four-year-old fiber joints surveyed during February and March, 1943.

Tables 44, 45, and 46, the data relating to fillers on each type and age of joint are considered with respect to the prevalence of high and low fillers at individual wheel paths.

From 7 to 45 per cent of the wheel paths over fiber, air-chamber, and bituminous premolded expansion joints had high fillers (Table 44). Where a direct comparison at equal ages can be made, the J-1 joints appear to have had a greater proportion of high fillers than the other types of metal joints, though the difference probably was not great enough to be of any real significance as far as riding comfort is concerned. There is a noticeable difference between the fiber and bituminous premolded joints of the same age, the latter showing about 50 per cent more high wheel paths. This may be explained by the fact that the bituminous premolded material extrudes greatly when compressed, while the extrusion of the fiber material is scarcely noticeable. On the basis of age, the wheel paths having high fillers ranged from 22 to 29 per cent, with a trend toward an increase with age, probably due to periodic filling and to progressive closing of the expansion joints. There was no significant difference in the average height of the high fillers for any of the air-chamber or premolded expansion joints.

Table 45 shows that the data for the contraction joints are in general agreement with those for the expansion joints. High fillers were found at 12 to 59 per cent of the wheel paths over these joints. There existed the same tendency for the proportion of high fillers to increase with age, and the average height of the high fillers was practically the same as for the expansion joints, approximately 1/16 in.

TABLE 45

SUMMARY SHOWING NUMBER OF WHEEL PATHS OVER CONTRACTION JOINTS HAVING HIGH FILLERS AND THE MAXIMUM AND AVERAGE HEIGHTS OF THE FILLERS

(Survey made during winter of 1939-40)

Age years	Type of Joint	Number of Joints Examined	Number of Wheel Paths Measured	Wheel Paths Having High Filler		Height of High Filler, in.	
				Number	Percentage	Maximum	Average
2	J-1	190	760	89	11.7	0.13	0.05
	J-4	77	308	74	24.0	0.16	0.04
Totals and Averages		267	1,068	163	15.4	....	0.05
3	J-1	175	700	236	33.7	0.12	0.04
	J-4	70	280	83	29.6	0.10	0.05
Totals and Averages		245	980	319	32.6	....	0.04
4	J-1	195	780	461	59.1	0.22	0.06
	J-2	100	400	184	46.0	0.26	0.05
	J-4	40	160	70	43.7	0.19	0.06
Totals and Averages		335	1,340	715	53.5	....	0.06
5	J-2	113	452	132	29.2	0.16	0.05

At the time of the examination of the 4-in. open joints and the 2-in. bituminous premolded joints, low fillers were much more prevalent than high fillers (Table 46). The wheel paths having high fillers ranged from 2.5 to 12.5 per cent and those having low fillers from 87.5 to 97.5 per cent, for the various age groups into which the joints were divided. The average extrusion was considerably less than the average intrusion or depression of the filler, the former ranging from 0.03 in. to 0.15 in. and the latter from 0.15 in. to 0.55 in. Obviously, at the time of year the examinations were made, low fillers in those joints were a greater factor in affecting riding quality than high fillers.

The data presented heretofore show that about 30 per cent of the wheel paths over fiber, 1-in. bituminous premolded, and air-chamber joints had high fillers. In other words, 30 per cent of the wheel paths were potentially rough, the degree of roughness depending on the height of extrusion. The extrusion averaged about 1/16 in. for the 1-in. bituminous premolded and air-chamber joints and about 1/8 in. for the fiber joints. These extrusions, about the same as may be expected of the fillers over transverse cracks, should contribute no more to roughness than do transverse cracks, except as their influence may be intensified at certain speeds by rhythmic build-up in the springs of vehicles due to the regular spacing of the joints. It should be remembered that the investigation was made at a time when the fillers were naturally low. Had the measurements been made when the joints were at their minimum seasonal opening, both the average height of the filler and the number of high wheel paths undoubtedly would have been considerably greater than shown by this report.

Considering the data for the 2-in. bituminous joints and the 4-in. open joints, all of the wheel paths were potentially rough, because these joints are wide enough for low fillers to be a factor in producing roughness. Low fillers were much more prevalent than high fillers. The average height of filler at wheel paths showing extrusion varied from 1/32 in. to 5/32 in., which is about the same as may be expected at transverse cracks. The average intrusion or depression ranged from 5/32 in. to 9/16 in. It should be remembered that at another season of the year the conditions would be entirely different. In hot weather, the closing of the joints and the swelling of the asphalt would result in most fillers being high, many of them to such an extent as to require the removal of some of the asphalt.

(2) Variations in Surface of Concrete. In making a study of surface variations, on the assumption that the pavements were built to the surface smoothness requirements in effect at the time of construction, the maximum variation in a 10-ft. length spanning a joint was taken as the element for comparison; namely, not more than 1/4 in. in 10 ft. for earlier pavements and not more than 1/8 in. in 10 ft. for pavement built after March 23, 1931. The latter specification covered all pavements less than eight years old at the time of the investigation. The data relating to surface variations are given in

TABLE 46

SUMMARY SHOWING NUMBER OF WHEEL PATHS OVER WIDE EXPANSION JOINTS HAVING HIGH AND LOW FILLERS, MAXIMUM AND AVERAGE HEIGHTS, AND MAXIMUM AND AVERAGE DEPRESSIONS

(Survey made during winter of 1939-40)

Age years	Type of Joint	Number Joints Examined	Number of Wheel Paths Measured	Wheel Paths Having High Fillers				Wheel Paths Having Low Fillers			
				Number	Percentage	Maximum height in.	Average height in.	Number	Percentage	Maximum depression in.	Average depression in.
4-5-6	4-in. open bituminous[1] premolded	40 10	160 40	19 2	11.9 5.0	0.21 0.06	0.06 0.03	141 36	88.1 90.0	1.28 0.31	0.26 0.15
Totals and Averages		50	200	21	10.5	....	0.06	177	88.5	....	0.24
7-8	4-in. open	45	180	15	8.4	0.19	0.11	165	91.6	1.26	0.48
9	4-in. open	49	196	8	4.1	0.53	0.15	188	95.9	2.50	0.55
10-11-12	4-in. open	30	120	3	2.5	0.06	0.05	117	97.5	2.78	0.35
16	4-in. open	10	40	5	12.5	0.17	0.12	35	87.5	0.58	0.33

[1] These joints are six years old.

Tables 47 to 52, inclusive. The variations included in these tables are those in the concrete surfaces, disregarding the joint fillers, the effect of which has already been discussed.

Tables 47 and 48, presenting data for expansion and contraction joints, respectively, show that the average maximum surface variations adjacent to these joints were for the most part only slightly greater than the limiting value permitted by the specifications in force at the time of construction. On the pavements with six-year-old bituminous premolded joints and four- to seven-year-old 4-in. open joints, the average variations exceeded the specification limits by about ⅛ in.; for all other pavements, the average variation exceeded the specification limit by less than 1/16 in. Neither age nor type of joint, except those mentioned above, appears to have any definite influence on surface variations. The four- to seven-year-old pavements with 4-in. open joints, built under the ⅛ in. requirement, showed approximately the same average maximum variations as the older pavements of this type built under

TABLE 47
SUMMARY SHOWING MAXIMUM VARIATIONS IN CONCRETE SURFACE ADJACENT TO EXPANSION JOINTS
(Survey made during winter of 1939-40)[1]

Age years	Type of Joint	Number of Joints	Average Maximum Variation in 10-ft. Length of Concrete Surface, in.				
			Wheel path				Average
			1	2	3	4	
1	Fiber	328	0.141	0.146	0.148	0.142	0.144
	Bituminous premolded	40	0.165	0.179	0.175	0.187	0.177
Totals and Averages		368	0.144	0.150	0.151	0.147	0.148
2	J-1	201	0.144	0.138	0.140	0.141	0.141
	J-4	77	0.132	0.126	0.126	0.135	0.130
Totals and Averages		288	0.136	0.130	0.131	0.135	0.133
3	J-1	166	0.142	0.137	0.138	0.148	0.141
	J-4	70	0.130	0.120	0.140	0.130	0.130
Totals and Averages		236	0.138	0.132	0.139	0.143	0.138
4	J-1	195	0.141	0.136	0.136	0.140	0.139
	J-2	100	0.131	0.146	0.143	0.146	0.142
	J-4	40	0.167	0.168	0.164	0.163	0.166
	Fiber	182	0.158	0.165	0.158	0.159	0.160
Totals and Averages		517	0.146	0.151	0.147	0.150	0.149
5	J-4	128	0.170	0.180	0.170	0.170	0.170
6	Bituminous premolded	10	0.238	0.280	0.289	0.287	0.275
4 to 7	4-in. open	92	0.210	0.200	0.200	0.230	0.210
8 to 16	4-in. open	150	0.240	0.230	0.230	0.240	0.240

[1] Four-year-old fiber joints surveyed during February and March, 1943.

TABLE 48
SUMMARY SHOWING MAXIMUM VARIATIONS IN CONCRETE SURFACE ADJACENT TO CONTRACTION JOINTS
(Survey made during winter of 1939-40)

Age years	Type of Joint	Number of Joints	Maximum Variation in 10-ft. Length of Concrete Surface, in.				
			Wheel path				Average
			1	2	3	4	
2	J-1 J-4	209 77	0.143 0.110	0.132 0.110	0.130 0.110	0.129 0.120	0.133 0.110
Totals and Averages		286	0.134	0.126	0.125	0.127	0.127
3	J-1 J-4	166 70	0.136 0.120	0.130 0.120	0.127 0.120	0.132 0.120	0.132 0.120
Totals and Averages		236	0.131	0.127	0.125	0.128	0.128
4	J-1 J-2 J-4	195 100 40	0.133 0.133 0.138	0.131 0.132 0.144	0.131 0.125 0.141	0.131 0.136 0.153	0.132 0.132 0.144
Totals and Averages		335	0.134	0.133	0.130	0.135	0.133
5	J-2	128	0.150	0.150	0.150	0.158	0.150

the ¼ in. in 10 ft. requirement. Both had average variations of about ¼ in. Data presented in Tables 47 and 48 indicate that joints may adversely affect riding qualities, but they do not reveal the condition at individual joints. It is necessary to make a more detailed study in order to obtain a clearer picture of the extent to which riding may be affected by joints.

Roughness, as indicated by its effect on passengers in a vehicle, is determined by the frequency and magnitude of the individual surface variations over which the vehicle passes. A large number of relatively small variations, if spaced at regular intervals, may result in a rhythmic build-up in the spring movements which intensifies the vertical movements of the vehicle until they become objectionable. Relatively large variations at infrequent intervals also are disturbing. Between these limits are many combinations of number and size of variations which will influence the riding qualities of pavements to various degrees.

Summarized data showing the size and frequency of the surface variations of pavements of various ages, considering expansion joints and contraction joints separately, are presented in Tables 49 and 50. In this analysis, it is assumed that pavements are smooth riding when built to a specification requirement that variations shall not exceed ⅛ in. in 10 ft.; in other words, that roughness varies directly with the number and size of variations which exceed ⅛ in. in 10 ft. Tables 49 and 50 give in percentages the wheel paths over joints which had surface variations of various sizes. For example, Table 49 shows that of the 1,292 wheel paths over 323 one-year-old fiber joints, 61.4 per cent had maximum variations of ⅛ in. or less, 34.4 per cent

TABLE 49

MAXIMUM VARIATIONS IN CONCRETE SURFACE ADJACENT TO EXPANSION JOINTS CLASSIFIED ACCORDING TO SIZE

(Survey made during winter of 1939-40)[1]

Age	Type of Joint	Number of Joints	Total Wheel Paths	Percentage of Wheel Paths Having Maximum Variation in 10 ft. of						Joints with Average Maximum Variation in 10 ft. Greater than 1/8 in.
years				Over 1/8-1/4 in.	Over 1/4-3/8 in.	Over 3/8-1/2 in.	Over 1/2 in.	Over 1/8 in.	1/8 in. and under	percentage
1	Fiber	323	1,292	34.4	3.9	0.3	0.0	38.6	61.4	47.4
	Bituminous premolded	40	160	43.8	5.6	1.9	0.0	51.3	48.7	72.5
Totals and Averages		363	1,452	35.4	4.1	0.5	0.0	40.0	60.0	50.0
2	J-1	201	804	36.3	3.6	0.5	0.0	40.5	59.5	49.2
	J-4	77	308	29.2	2.0	0.3	0.0	31.5	68.5	37.7
Totals and Averages		278	1,112	34.3	3.2	0.5	0.0	38.0	62.0	46.0
3	J-1	166	664	36.1	2.6	0.3	0.0	38.0	61.0	60.3
	J-4	70	280	31.4	2.1	0.4	0.0	33.9	66.1	40.0
Totals and Averages		236	944	34.7	2.5	0.3	0.0	37.5	62.5	54.3
4	J-1	195	780	29.7	3.2	0.3	0.0	33.2	66.8	46.2
	J-2	100	400	42.8	1.7	0.0	0.0	44.5	55.5	55.0
	J-4	40	160	35.6	8.8	1.2	0.0	45.6	54.4	67.5
	Fiber	182	728	44.2	22.5	3.3	0.5	56.7	43.3	60.9
Totals and Averages		517	2,068	37.8	10.1	1.4	0.2	44.7	55.4	54.7
5	J-2	118	472	48.3	8.1	1.0	0.0	57.4	42.6	79.7
6	Bituminous premolded	10	40	35.0	45.0	10.0	0.0	90.0	10.0	100.0
4-7	4-in. open	92	368	43.2	12.2	4.9	4.6	64.9	35.1	72.8
8-16	4-in. open	150	600	48.3	19.5	5.9	3.0	77.2	22.8	90.7

[1] Four-year-old fiber joints surveyed during February and March, 1943.

TABLE 50

MAXIMUM VARIATIONS IN CONCRETE SURFACE ADJACENT TO CONTRACTION JOINTS CLASSIFIED ACCORDING TO SIZE

(Survey made during winter of 1939-40)

Age years	Type of Joint	Number of Joints	Total Wheel Paths	Percentage of Wheel Paths Having Maximum Variation in 10 ft. of						Joints with Average Maximum Variation in 10 ft. Greater than ⅛ in. percentage
				Over ⅛-¼ in.	Over ¼-⅜ in.	Over ⅜-½ in.	Over ½ in.	Over ⅛ in.	⅛ in. and under	
2	J-1 J-4	199 77	796 308	31.6 21.1	2.8 0.7	0.4 0.0	0.1 0.0	34.9 21.8	65.1 78.2	43.2 28.6
Totals and Averages		276	1,104	28.7	2.2	0.3	0.1	31.2	68.8	39.1
3	J-1 J-4	166 70	664 280	27.8 21.4	2.0 1.8	0.0 0.4	0.0 0.0	29.8 23.6	70.2 76.4	42.9 22.9
Totals and Averages		236	944	25.9	1.9	0.1	0.0	28.0	72.0	37.0
4	J-1 J-2 J-4	195 100 40	780 400 160	28.6 37.5 35.0	1.8 1.0 3.8	0.4 0.0 0.6	0.0 0.0 0.0	30.8 38.5 39.4	69.2 61.5 60.6	42.6 52.0 47.5
Totals and Averages		335	1,340	32.0	1.8	0.3	0.0	34.1	65.9	46.2
5	J-2	113	452	41.2	3.8	0.4	0.2	45.6	54.4	58.4

over ⅛ in. and not more than ¼ in., 3.9 per cent over ¼ in. and not more than ⅜ in., and 0.3 per cent over ⅜ in. and not more than ½ in.

Variations over ⅛ in. were found on from 22 to 90 per cent of the wheel paths measured over expansion and contraction joints. There is little indication that age or type of air-chamber expansion joints had any influence on the number of variations over ⅛ in. Metal contraction joints had a larger percentage of wheel paths with maximum variations of ⅛ in. or less. The pavements having the greatest percentage of variations at joints over ⅛ in. were those constructed with bituminous premolded and 4-in. open joints. Approximately 51 per cent of the wheel paths over the one-year-old bituminous premolded joints had variations greater than ⅛ in. Ninety per cent of the wheel paths over the six-year-old bituminous joints were in the same category. These, however, were special joints installed experimentally on one section of pavement and, as only 10 joints were examined, the data should not be given too much weight. A greater percentage of wheel paths over 4-in. open joints in pavements eight to 16 years old showed maximum variations exceeding ⅛ in. than over the same type of joint in pavements four to seven years old, but this may be due to the older pavements being built to a less strict surface requirement rather than to age.

In Table 51, relative roughness ratings are given for pavements of various ages with different types of joints. The relative roughness rating is a value determined by multiplying the percentage of wheel paths in the several maximum height increments by arbitrarily assigned constants and adding these products together. In this analysis, the increment between ⅛ in. and ¼ in. is given a value of one, that between ¼ in. and ⅜ in. a value of two, that between ⅜ in. and ½ in. a value of three, and variations over ½ in., of which there were very few, are given a value of four. In analyzing these ratings, it must be remembered that they are relative values which have no quantitative significance. In other words, if one pavement has a rating of 100 and another a rating of 50, the former is rougher than the latter, but not necessarily twice as rough.

In Table 51, the maximum and minimum averages shown are for the sections of pavement in each classification having the highest and lowest ratings. The range is simply the difference between maximum and minimum averages. The grand average is the weighted average for all the sections in one classification; for example, this value for the one-year-old fiber joints is the average for 33 sections of pavements weighted in accordance with the number of joints examined on each section. The last three columns give in percentages the number of sections whose average roughness rating falls in three different ranges. The reasons for choosing these ranges will be discussed later.

The maximum, minimum and grand averages are generally favorable to the air-chamber joints, the values for the premolded and 4-in. open joints being in most cases somewhat higher. There are no indications that age influenced roughness ratings. While the pavements with 4-in. open joints appear to be an exception, those 8 to 16 years old showing higher ratings than do those from 4 to 7 years old, it must be remembered that the former were built to a less strict surface requirement, which may easily account for the difference.

Considering individual sections of pavement, an analysis may be made which correlates to some extent roughness rating in terms of riding quality. Route 161, Section 10, a one-year-old pavement with fiber joints, has a roughness rating of 102.5. Section 10-H, which adjoins Section 10, is approximately the same age and has a roughness rating of 47.5. A group of engineers driving over both these sections one day at the same speed detected

TABLE 51
SUMMARY OF ROUGHNESS RATINGS FOR PAVEMENT SURFACES ADJACENT TO EXPANSION AND CONTRACTION JOINTS
(Survey made during winter of 1939-40)[1]

Age years	Type of Joint	Roughness Rating				Percentage of Sections with Average Rating between		
		Maximum average	Minimum average	Grand average	Range	0-50	51-100	Over 100
1	Fiber	130.0	2.5	43.1	127.5	64	30	6
	Bituminous premolded	110.0	25.0	60.7	85.0	50	25	25
Average		127.9	4.9	45.0	122.9	62.4	29.4	8.1
2	J-1 (E)	115.0	0.0	45.3	115.0	60	30	10
	J-1 (C)	87.5	0.0	38.8	87.5	65	35	0
	J-4 (E)	82.5	2.5	34.1	80.0	75	25	0
	J-4 (C)	62.5	0.0	22.5	62.5	88	12	0
Average		93.2	0.3	38.2	92.9	67.7	28.7	3.6
3	J-1 (E)	87.5	2.5	42.2	85.0	72	28	0
	J-1 (C)	100.0	2.5	31.8	97.5	72	28	0
	J-4 (E)	82.5	12.5	36.8	70.0	71	29	0
	J-4 (C)	52.5	2.5	26.2	50.0	86	14	0
Average		86.0	4.0	35.4	82.0	73.9	26.1	0
4	J-1 (E)	95.0	2.5	37.0	92.5	75	25	0
	J-1 (C)	95.0	5.0	33.4	90.0	80	20	0
	J-2 (E)	82.5	0.0	46.2	82.5	40	60	0
	J-2 (C)	60.0	0.0	39.5	60.0	70	30	0
	J-4 (E)	92.5	22.5	56.8	70.0	50	50	0
	J-4 (C)	82.5	5.0	44.4	77.5	75	25	0
	Average	87.0	3.8	39.3	83.2	69.0	31.0	0
	Fiber	165.0	25.0	70.5	140.0	33	45	22
Average		96.1	8.6	47.0	87.5	63.9	33.0	3.1
5	J-2 (E)	110.0	17.5	67.5	92.5	25	58	17
	J-2 (C)	112.5	2.5	50.8	110.0	42	50	8
Average		111.3	10.0	59.2	101.3	33.5	54.0	12.5
6	Bituminous premolded	155.0	155.0	155.0	0	0	0	100
4-7	4-in. open	200.0	45.4	100.8	154.6	25	25	50
8-16	4-in. open	207.5	75.0	117.3	122.5	0	42	58

[1] Four-year-old fiber joints surveyed during February and March, 1943.
(E)-Expansion Joint (C)-Contraction Joint

a marked difference in the riding qualities of the two sections. On Section 10 every joint was noticeable to a disturbing degree by the pitching of the car, while Section 10-H was smooth riding and produced no indications of the presence of joints. Unfortunately, no similar comparisons have been made for other pavements for which the roughness rating is known, but judging from this one experience it seems probable that a pavement having a roughness rating around 50 will be smooth riding, while one having a rating around 100 will be rough riding. If it is assumed that pavements having a roughness rating of 50 or less are smooth, those with ratings between 50 and 100 are moderately rough, and those with rating above 100 are very rough, a further basis for comparison is provided.

Attention is called to the fact that these ratings are not real measures of pavement roughness, since they do not take into consideration the spacing of joints or the roughness of the pavement between joints induced by transverse cracks or other causes. In the case of the example given above, the two sections being of equal age and having the same type of joint at the same spacing, it is probable that the ratings provide a reasonable comparison of surface smoothness. However, if one pavement has joints at 50-ft. intervals and another has joints at 1,000-ft. intervals, their respective riding qualities probably will be distinctly different in spite of the fact that the roughness ratings of the joints may be equal. Actually the roughness rating is a relative measure of the influence of joints on roughness, without regard to spacing or other factors.

In the last three columns of Table 51, the percentage of sections in each of these three roughness classifications is given for each type and age of joint. On the whole, these data show that the pavements with air-chamber joints had slightly lower ratings than those with premolded and 4-in. open joints, a greater proportion of the former falling in the lower rating classifications. Of the pavements with premolded and 4-in. open joints, those containing fiber joints had the highest percentage of sections in the smooth classification; those with bituminous premolded joints, exclusive of one special section six years old, were next; and those with 4-in. open joints had the highest percentage in the rougher classifications. In the case of the latter type, the older pavements are the rougher, but, as explained before, this may be due to the less rigid surface requirement under which they were built.

There is no indication from these data that age affects the distribution among the three roughness classifications. This is evidenced by the two-year-old pavements with air-chamber joints having averages of 67.7, 28.7, and 3.6 per cent in the three classifications, respectively; three-year-old pavements 73.9, 26.1, and 0.0 per cent, respectively; and four-year-old pavements 69.0, 31.0, and 0.0 per cent, respectively.

A study of the data shows that in general there was no important difference between the roughness classifications for the pavements with J-1 and J-4 joints, one showing to advantage at one age and the other having a greater percentage in the smooth classification at another age. The pavements with

J-2 joints, especially those five years old, had proportionately fewer sections in the smooth classification and commensurately higher percentages in the rougher classifications.

Two reasons are advanced to explain these findings. First, those pavements were the initial ones built with J-2 joints, and the lack of experience in installing the joints, together with the fact that the joint, being in the development stage, had many faults, may have resulted in roughness being built into the pavements. Second, the nature of the load transmission device on this joint is such that any misalignment of the devices, or deposits of mortar between the base sleeve and the devices, will tend to move the ends of the slab vertically as the joints open and close due to changes in length of the concrete.

The load transmission device consists of sections of structural angle about 1 ft. long, placed alternately on each side of the joint, with the vertical leg extending upward and secured to the end of the concrete slab by means of lugs, and the horizontal leg extending across the joint and under the end of the adjacent slab. The horizontal legs are encased in a sheet metal sleeve to provide expansion space in which the angles can move. The horizontal legs must remain parallel to the plane of the pavement in order to act properly. If they become tilted during construction, as they often did, then the bottom of the slab conforms to the slope of the horizontal leg, and during movement of the pavement the ends of the slabs tend to ride up on the angle, causing them to rise. Uneven deposits of mortar between the angle and the base sleeve will produce the same result.

As shown by other analyses, these data also show that pavement surfaces adjacent to contraction joints are smoother than those adjacent to expansion joints. For example, the pavements two years old with J-1 expansion joints had 60, 30, and 10 per cent in the smooth, moderately rough, and very rough classifications, respectively, and those with J-1 contraction joints had 65, 35, and 0.0 per cent in the same classifications. The same relative comparisons exist for the other types of joints.

The foregoing analyses indicate that as a whole the concrete surfaces adjacent to joints have greater variations than permitted by the specifications under which the pavements were built. The wheel paths having maximum variations in 10 ft. of more than ⅛ in. ranged from approximately 22 per cent for pavements with one type of contraction joint to 90 per cent found on one section of pavement with 2-in. bituminous premolded joints. The great majority of variations were not over ¼ in. The pavements with air-chamber joints in general had smaller variations and fewer exceeding the limits of the specifications than pavements with other types of joints. The surfaces adjacent to metal contraction joints were somewhat less rough than those adjacent to air-chamber expansion joints. Where direct comparisons are possible, it appears that age had no effect on the surface variations at joints.

In 1943 measurements of surface elevations were made at 10 expansion joints and 10 contiguous transverse cracks on each of seven sections containing fiber joints, 10 sections containing air-chamber joints, and 10 sections containing 4-in. open joints. The relative roughness ratings for the joints and cracks for each section are given in Table 52. In almost every case the roughness rating for the surface adjacent to cracks was considerably lower than for joints. Comparing averages, fiber joints had a roughness rating of 51.8, contiguous cracks a rating of 20; air-chamber joints a rating of 48.5 as compared with 29.3 for the cracks; 4-in. open joints a rating of 135.9, adjacent cracks 44.3. It is evident from these data that concrete surfaces adjacent to expansion joints are subject to greater variations than those at transverse cracks and that, on the whole, jointed pavements can be expected to be somewhat less smooth than cracked unjointed pavements.

Even though this difference between joints and transverse cracks did not exist, pavements with a relatively large number of joints may be expected to be rougher than unjointed pavements or pavements with joints at long intervals, at least for a greater part of their service life, because of the greater frequency of joints. While it is true, as is shown hereinafter, that the number of cracks in an unjointed pavement, or the number of joints and cracks in a pavement with joints at long intervals, eventually may equal the number of joints and cracks in a pavement of equal age built with joints at close intervals, such as those in Illinois with air-chamber joints, on the average this will not occur until well along in the service life of the pavement. Based on its statewide survey, the University committee, in its report to the Governor, concluded, "There are indications that age for age the pavements in Illinois

TABLE 52
COMPARATIVE RELATIVE ROUGHNESS RATINGS OF SURFACES ADJACENT TO EXPANSION JOINTS AND TRANSVERSE CRACKS FOR VARIOUS PAVEMENT SECTIONS
(Measurements made during April, 1943)

Fiber Joint Sections		Air-Chamber Joint Sections		4-in. Open Joints	
Average at joints	Average at cracks	Average at joints	Average at cracks	Average at joints	Average at cracks
100.0	25.0	85.0	17.5	92.5	47.5
35.0	25.0	15.0	25.0	135.0	65.0
27.5	15.0	30.0	0.0	130.0	42.5
102.5	22.5	32.5	47.5	192.0	55.0
142.5	22.5	67.5	25.0	102.5	35.0
0.0	0.0	55.0	47.5	110.0	27.5
55.0	30.0	22.5	15.0	65.0	32.5
.....	....	12.5	20.0	90.0	30.0
.....	....	85.0	42.5	252.5	47.5
.....	....	80.0	52.5	189.0	60.0
Average 51.8	Average 20.0	Average 48.5	Average 29.3	Average 135.9	Average 44.3

Each represents one section of pavement.
Average ages of pavements: fiber joint sections—5 years; air-chamber joint sections—6 years; 4-in. open joint sections—11 years.

built with 4-in. joints at 800-ft. intervals are better riding and better appearing than those built under the present specifications."[11]

As will be shown later in this report, unjointed pavements may be 12 or 13 years old before they develop transverse cracks in sufficient number to equal joints spaced at 30-ft. intervals. Hence during this 12- or 13-year period, when the motoring public expects the most in riding comfort, the unjointed pavements and those with joints at long intervals can be expected to be smoother riding than pavements with closely spaced joints. This seems to be the most important factor in the consideration of joints with respect to riding quality; that is, when joints are installed, especially at close intervals, roughness results which on the average will not be equalled in an unjointed pavement for many years. Therefore, in order to maintain good riding qualities for the longest possible time, it would seem logical to install the smallest number of joints commensurate with other design requirements.

(h) Cracking of Concrete Pavements

When the use of air-chamber expansion and metal contraction joints was adopted, it was thought that spacing them at intervals of 30 ft. would eliminate to a great extent the formation of transverse cracks. This belief was based on the results of crack surveys on a large mileage of pavement built without joints or with 4-in. open joints spaced at long intervals. The average crack interval in pavements 13 years old was approximately 28 ft., and extrapolation of the curve plotted from these data indicated that new cracks would develop very slowly after that age. Subsequent experience, however, proved that joints spaced at 30 ft. will not control cracking. A survey made in 1934 of 280 miles of pavement built in 1933, with air-chamber expansion and metal contraction joints spaced at 30 ft., showed that cracks developed in more than 2 per cent of the panels between joints the first year. A survey made in 1937 of some 50 miles of these pavements showed that 17 per cent of the panels were cracked at four years.

The survey in 1939 included 606 miles of pavement, consisting of 520 miles with joints spaced 30 to 50 ft., ranging in age from one to six years, and 86 miles with 4-in. open joints ranging in age from four to 16 years. In making the survey on pavements containing fiber, bituminous premolded, and air-chamber joints, the cracks were plotted with reference to adjacent joints, and such features as cut, fill, cut and fill, culverts, bridges, drainage, subgrade replacement, and mudjacked areas were recorded. This gives a complete record which will serve as a basis for a study of progressive cracking in the future. On the pave-

[11] Air-chamber expansion joints at 90 ft. with two metal contraction joints between, dividing the pavement into 30-ft. panels.

ments with 4-in. open joints, the number of cracks was counted and the average crack interval determined by dividing the length of the pavement by the number of cracks. Summaries of the data are given in Tables 53 and 54.

Referring to Table 53, which gives data for pavements with air-chamber and premolded joints, it is seen that the percentage of broken panels increases with age, ranging from 7.7 per cent at one year to 32.4 per cent at five years. The cracking in the six-year-old pavement is abnormally high when compared with that at other ages, but only one section of pavement was surveyed and this perhaps was not representative of average conditions. As shown by the table, the amount of cracking on individual sections varied considerably from the average, regardless of age of pavement or type of joint; it is probable that the cracking found on this section of six-year-old pavement is more nearly representative of the maximum condition to be expected. There is no indication from the average data in Table 53 that one type of joint influenced cracking more than another. The maximum cracking on individual sections was somewhat lower in the pavements with J-1 joints than in pavements with other types of joints, but considering the many variables which may affect cracking, this probably is of no significance.

From the available data it appears that cracks develop more rapidly in reinforced pavements with joints spaced at 50-ft. intervals than in unreinforced pavements with joints 30 ft. apart. Table 53 shows that there were 8.5 transverse cracks per mile in the one-year-old reinforced pavements with fiber joints at 50-ft. intervals, whereas there were 4.5 cracks per mile in the one-year-old unreinforced pavements with bituminous premolded joints spaced 30 ft. apart. At four years, the unreinforced pavements with air-chamber expansion and metal contraction joints at 30 ft. had approximately 40 natural cracks per mile, as compared with 55 cracks per mile in pavements with fiber joints at 50-ft. intervals. A survey made in 1934, covering one-year-old unreinforced pavements with joints spaced 30 ft. apart, showed 4.0 cracks per mile, which agrees closely with the value given above for pavements with bituminous premolded joints. The only apparent difference between the pavements with fiber joints and those with bituminous premolded joints is that the former contain wire mesh reinforcement and the joints are spaced 50 ft. apart. It does not seem reasonable that the mesh reinforcement would contribute to cracking; however, it may be that the warping stresses in a 50-ft. slab are greater than in one 30 ft. long. The data are too limited to warrant a definite conclusion.

TABLE 53

GENERAL SUMMARY OF DATA RELATING TO CRACKING OF CONCRETE PAVEMENTS BUILT WITH PREMOLDED AND METAL JOINTS

(Survey made during winter of 1939-40)[1]

Age years	Type of Joint	Length of Pavement ft.	Total Number of Panels between Joints	Total Broken Panels in Percentage of Total Panels			Broken Panels in Percentage of Total Number Broken		Number of Natural Cracks per mi.	Number of Cracks and Joints per mi.	Number of Corner Breaks per 100 mi.	
				All sections	Maximum	Minimum	Between expansion and contraction joints	Between contraction joints			Exterior	Interior
1	Fiber	564,164	11,224	8.1	63.5	0.0	....	....	8.5	115	1	0
	Bituminous premolded	25,686	854	2.6	3.7	2.3	....	....	4.5	182	0	21
Totals and Averages		589,850	12,078	7.7	....	...	....	....	8.3	117	1	1
2	J-1	468,010	15,622	13.5	68.2	0.5	66.8	33.2	23.8	203	3	16
	J-4	211,650	7,055	21.6	99.0	0.0	66.7	33.3	38.0	211	17	27
Totals and Averages		679,660	22,677	16.0	....	...	66.8	33.2	28.2	203	8	19
3	J-1	297,030	9,901	24.0	49.0	2.2	66.9	33.1	42.2	220	14	34
	J-4	156,180	5,278	19.0	60.5	0.8	63.7	36.3	33.8	211	7	30
Totals and Averages		453,210	15,179	22.2	....	...	65.9	34.1	39.4	220	12	33
4	J-1	377,740	12,184	24.6	55.9	6.5	67.9	32.1	41.9	211	46	75
	J-2	303,390	10,113	21.4	60.7	1.0	68.0	32.0	37.7	211	101	66
	J-4	53,070	1,769	22.4	62.6	3.5	70.3	29.7	39.4	211	30	40
	Fiber	382,841	7,222	33.6	94.3	0.0	....	....	55.1	162	21	32
Totals and Averages		1,117,041	31,288	25.5	....	...	68.0	32.0	45.2	194	52	56
5	J-2	256,260	8,542	32.4	64.3	3.8	67.9	32.1	56.8	230	27	54
6	Bituminous premolded	25,087	1,591	85.6	[2]	[2]	....	....	293.3	660	21	253

[1] Four-year-old fiber joints surveyed during February and March, 1943.
[2] Only one section included.

It was noted during the 1939 investigation that the cracks in pavements with wire mesh reinforcement were not opening up, even in cold weather. This indicated that the wire mesh was effective in holding the cracked slabs together; however, the pavements, being only one year old at the time, had not been in service long enough to establish the real merit of mesh reinforcement. Special attention was given this question in the supplementary investigation made in 1943 on five-year-old pavements with mesh reinforcement. On the 18 sections of pavement examined, 3,997 transverse cracks were found and all but 32 were tightly closed. The open cracks were isolated in three sections, one in each of two sections and 30 on the third section. The open cracks in the latter section were not distributed throughout the entire length but had developed on only one part of the section, indicating that they probably were caused by some condition peculiar to that part of the section. Examinations made throughout the state in 1945 indicated that the wire mesh reinforcement in pavements built in 1938 was still highly effective in holding cracks tightly closed. It was also observed that the expansion joints in the reinforced pavement had not exhibited the tendency to close permanently as do joints in unreinforced pavements. At four years the air-chamber joints, which were in unreinforced pavements, had closed an average of 0.35 in. to 0.50 in., while fiber joints in reinforced pavements of the same age were practically the same width as when installed (Table 41).

These data show conclusively that the wire mesh reinforcement was effective in holding transverse cracks closed. It is possible that corrosion may eventually so reduce the cross section of the wires where they are exposed at cracks that they will no longer have the strength to resist the forces tending to pull the slabs apart. If that occurs, then, of course, the pavements will act as any other pavement. Foreign material will enter the cracks when they open up and the joints will close progressively until their value in absorbing temperature movements of the concrete will be destroyed. Should future investigations of these pavements reveal that corrosion seriously affects the value of wire mesh, then the need for a heavier mesh would be indicated.

Based on present knowledge, however, it would appear that mesh weighing 55 lb. per 100 sq. ft. in 50-ft. panels is highly effective in holding broken slabs together, thereby preventing to a large extent the growth of pavements caused by infiltration of foreign material into cracks, and extending the value of expansion joints over a longer period.

There was a wide range in the amount of cracking found on individual sections of pavement in the same age group containing the same kind of joints. With the exception of one-year-old pavements with bituminous premolded joints, which had very few broken panels, the maximum number of broken panels on any one section ranged from 49 to 99 per cent and the minimum number from 0.0 to 6.5 per cent. Such differences as exist between the data for different types of joints are negligible and cannot be considered as indicating that one type of joint had more influence on cracking than another type, as long as other conditions were the same.

The rate at which transverse cracks occur is indicated by the average number of cracks per mile. It will be seen from Table 53 that the number of transverse cracks increased with the age of the pavement. It also appears that more transverse cracks occurred during some years than others. In one district where progressive surveys were made for several years on seven sections of pavement whose aggregate length was 30 miles, it was found that in the period between February and December, 1939, the number of transverse cracks increased at the rate of 53 per mile, as compared with an increase of two per mile during the period from February, 1937, to February, 1939.

Some authorities are of the opinion that pavements will crack less if they are under longitudinal compression, the theory being that the compressive stress must be overcome before the slab can develop the tensile stresses that cause the cracks. The air-chamber expansion joints were placed 90 ft. apart with two contraction joints between each pair of expansion joints. This arrangement gave one slab between two contraction joints and two slabs bounded by an expansion joint and a contraction joint. It was suggested that if this arrangement provided greater compression between contraction joints than between the expansion and contraction joints, fewer cracks could be expected between contraction joints. As shown by Table 53, it was found that almost exactly one-third of the broken panels were between contraction joints. Since one-third of all the panels were between contraction joints, it is evident that the same relative cracking occurred in both types of panels. This does not necessarily refute the theory referred to above. Relief no doubt was afforded the center panel equally as much as the two panels adjacent to the expansion joints, resulting in all being under the same stress.

The results of the survey on pavements built with 4-in. open joints are given in Table 54. It is seen from these data that the number of transverse cracks increases with the age of the pavement. The trend, however, is not so definite as in the case of the data obtained from pavements built with metal and premolded joints at close spacings,

TABLE 54
SUMMARY OF DATA RELATING TO CRACKING OF CONCRETE PAVEMENTS BUILT WITH 4-IN. OPEN JOINTS
(Survey made during winter of 1939-40)

Age years	Length of Pavement ft.	Number of 4-in. Joints	Average Distance between Joints ft.	Number of Natural Cracks	Number of Natural Cracks per mi.	Number of Cracks and Joints per mi.	Number of Corner Breaks per 100 mi.	
							Exterior	Interior
4	33,371	95	355	303	48	63	111	111
6	37,657	53	710	946	132	139	0	89
7	18,885	24	821	224	63	69	28	252
8	92,118	109	846	2,603	151	155	115	120
9	182,977	208	880	6,996	203	211	276	440
10	15,591	44	363	311	106	120	440	339
11	61,481	90	683	1,748	151	160	258	463
16	13,985	14	1,075	630	238	243	340	982

possibly because the lengths of pavements included in each age group in Table 54 are not sufficient to give good averages. The survey included from 4 to 35 miles of pavement having 4-in. open joints in each age group, as compared with from 50 to 100 miles of pavements with joints at close spacings, which probably accounts for the more uniform data in the latter case. The data agree substantially with the results of earlier surveys, which indicated that the average length of unbroken slabs at 13 years was about 28 ft.

It appears from Table 53 that serious transverse cracking on pavements with joints spaced at 30 ft. began during the second year and increased during subsequent years. At the end of four years there were almost as many transverse cracks per mile in these pavements as there were in pavements of the same age built with 4-in. open joints at 800 to 1,000 ft. This may not be a true comparison, because the data for pavements with 4-in. open joints do not establish definitely the probable average spacing of transverse cracks for various ages. However, it is established definitely that cracks occur rather frequently in 30-ft. panels even at early ages, and it is reasonable to expect that eventually all such panels will develop at least one crack, giving a maximum average spacing of 15 ft., or a total of 352 joints and cracks per mile. This is substantially in agreement with the results of investigations conducted by the Public Roads Administration (*Public Roads,* November, 1935), and the theoretical analysis made by Dr. H. M. Westergaard (*Public Roads,* June, 1929), both of which indicate that stresses due to restrained warping are relieved materially in slabs 10 to 15 ft. long. Pavements with 4-in. open joints which were 16 years old at the time of this survey had a total of 243 joints and cracks per mile, or an average crack interval of 22 ft. (Table 54).

It appears, therefore, that maintenance of transverse cracks will in time be equal for pavements with joints at 30 ft. and those with 4-in. open joints spaced 800 to 1,000 ft. When maintenance of the joints is also considered, especially during the later years of the pavement's serviceable life, when the joints unquestionably will require as much or more attention than transverse cracks, the cost of adequately maintaining the pavements with joints at 30 ft. may exceed that for pavements with 4-in. open joints.

Perhaps the most serious aspect of cracks in pavements built with air-chamber expansion joints and metal contraction joints during the period between 1933 and 1938 is that the cracks have no provision for transfer of load between the exterior corners thus formed. This does not apply to the pavements built prior to 1933, because they were built with marginal edge bars which are known to provide some load transfer. It also is not considered serious in the case of pavements built in accordance with the design adopted in 1938, which includes wire mesh reinforcement; this is expected to provide load transfer by the fact that the steel will hold the slabs together so as to maintain aggregate interlock. The cracks in pavement without marginal bars or mesh reinforcement form free edges, epecially during periods of contraction when the cracks are open sufficiently to destroy aggregate interlock. These cracks are points of potential weakness, and it seems reasonable to expect numerous corner breaks and other load failures as the pavements become older.

During recent years "pumping" at joints and transverse cracks has become a serious problem on pavements on some types of soil carrying heavy truck traffic, especially when drainage and subgrade conditions are such that a high water table is maintained. It appears that transverse cracks without load transfer features would be especially susceptible to this action, because the ends of the slab on each side of the crack deflect independently and the deflections will be larger than if the slabs had mutual support. No data to substantiate this opinion were obtained from these investigations, possibly because such information was not requested specifically. Cases are known, however, where "pumping" is a serious problem at air-chamber joints. It is entirely possible that most of the pavements included in this survey were not old enough to have developed this condition and that such failures will show up to an increasing degree as the pavements become older.

It was stated above that corner breaks and other load failures might be expected in pavements in which no provision had been made for load transmission at future transverse cracks. As yet, relatively few corner breaks have occurred in the pavements with closely spaced joints, as is seen from Table 53. Since all load failures seem to be a function of time, the small number of corner breaks may be due to the fact that these pavements are still relatively young. The data, however, show a definite increase with age in both exterior and interior corner breaks and some indication that they form more rapidly as the pavements become older. For example, exterior and interior corner breaks each occurred at the rate or one per hundred miles in the pavements one year old; eight and 19 per hundred miles, respectively, in the pavements two years old; 12 and 33 per hundred miles, respectively, in the pavements three years old; and 68 and 69 per hundred miles, respectively, in the pavements four years old. Interior corner breaks occurred much more frequently than exterior corner breaks. This was true of almost every age and joint group into which the data were divided. Considering all the pavements with closely spaced joints as a whole, the interior corner breaks exceeded the exterior corner breaks by almost 46 per cent, there being 188 of the former and 129 of the latter. This amounts approximately to one interior corner break every 3.0 miles and one exterior corner break every 4.7 miles.

Obviously, the frequency of these failures is not serious at the present time. An indication of how these failures may increase as the pavements become older may be obtained from the data for pavements with 4-in. open joints (Table 54). It has been pointed out that pavements in which air-chamber joints were installed will sooner or later develop as many cracks as pavements in which 4-in. open joints have been installed; also, that the cracks which develop between the air-chamber joints will probably be more susceptible to load failures because of the absence of load transfer. It may be concluded then that pavements with joints at 30-ft. intervals and no wire mesh reinforcement, marginal bars, or other provision for load transfer across transverse cracks, will in time develop as many or more corner breaks than pavements with 4-in. open joints.

The number of exterior and interior corner breaks in the pavements with 4-in. open joints ranged from 89 to 1,322 per hundred miles, depending on age (Table 54). Interior corner breaks occurred more frequently than exterior corner breaks, the former being about 62 per cent greater than the latter. This was true of every age group but one. Considering the 86 miles of pavement included in this survey, there were 156 exterior and 253 interior corner breaks, or one exterior failure for every 0.55 miles and one interior break for every 0.34 miles. In the pavements 11 years old, there was one exterior and one interior corner break every 0.39 miles and 0.21 miles, respectively. Corresponding values for pavements 16 years old were one exterior corner break every 0.29 miles and one interior break every 0.1 mile. Looking at this another way, on the 16-year-old pavements there were a total of 646 joints and cracks, making a total of 2,584 exterior and interior corners each, of which about 0.33 per cent of the exterior corners and 1.0 per cent of the interior corners had failed. The section of 16-year-old pavement covered by the survey is on a federal route carrying approximately 1,250 vehicles per day.

These figures give an indication of the number of corner failures which may occur eventually in those pavements built with joints at 30-ft. intervals, without provision for transfer of load across intermediate transverse cracks. Due consideration should be given, however, to the fact that these newer pavements are of heavier design and are therefore capable of withstanding greater loads. Most of the pavements with 4-in. open joints are of the 9-6-9 design, and all of the pavements with joints at 30-ft. intervals are of the 9-6½-9 or heavier design.

It is not certain, however, that the heavier design will offset the lack of load transfer at transverse cracks. The frequency with which corner failures may be expected, as shown by the survey of old pavements, is small when compared with the total number of corners, and it does not appear that corner failures in these pavements will present a serious maintenance problem as long as the volume of heavy traffic and the maximum allowable loads are not increased. If traffic conditions should change, even the heavier designs may not be sufficient, because experience shows that the present allowable loads border on the critical.

The data obtained from the survey of pavements with fiber, bituminous premolded, and air-chamber joints are arranged in Table 55 to show the relative cracking of pavement which followed the existing grade closely, and pavement on fills, in cuts, in cut and fill, and over culverts. The number of broken panels in each of these classifications is given in percentages of the total number of panels in the classification. It appears that there was somewhat less cracking in slabs at grade and considerably more cracking in slabs over culverts. The greater percentage of cracked panels over culverts was especially evident in the older pavements; 42.4, 51.2, and 53.7 per cent of all panels over culverts were broken on the three-, four-, and five-year-old pavements, respectively. Except for the six-year-old pavement, the percentage of failures over culverts was approximately twice as great as at each of the other locations. The six-year-old pavement consists of only one section, which had abnormally high percentages of failures at all locations and probably represents a maximum condition rather than an average. Panels on fills, in cuts, and in cut and fill had about the same proportionate number of failures.

TABLE 55
SUMMARY SHOWING INFLUENCE OF LOCATION OF PAVEMENT WITH RESPECT TO SURROUNDING TOPOGRAPHY AND CULVERTS ON CRACKING OF PAVEMENTS BUILT WITH PREMOLDED AND METAL JOINTS
(Survey made during winter of 1939-40)[1]

Age years	Type of Joint	Broken Panels in Percentage of Total Panels				
		At grade	On fills	In cuts	Cut and fill	Over culverts
1	Fiber	12.4	6.5	8.7	11.4	8.8
	Bituminous premolded	2.0	1.3	8.9	14.3	7.7
Weighted Averages		10.2	6.3	8.7	11.5	8.8
2	J-1	16.6	13.3	12.5	15.5	21.4
	J-4	15.0	28.0	13.4	14.4	21.8
Weighted Averages		16.1	21.9	12.8	15.3	21.5
3	J-1	17.7	21.5	33.7	22.6	42.9
	J-4	19.9	16.4	23.3	30.7	41.1
Weighted Averages		18.5	19.6	30.3	25.2	42.4
4	J-1	21.7	25.6	23.6	27.8	57.7
	J-2	12.7	22.2	23.7	18.1	50.8
	J-4	12.5	23.1	33.9	9.7	25.0
	Fiber	35.7	30.3	39.6	26.9	45.4
Weighted Averages		23.4	25.4	26.9	23.5	51.2
5	J-2	30.8	28.8	38.2	23.7	53.7
6	Bituminous premolded	86.4	82.8	91.8	93.9	97.2

[1] Four-year-old fiber joints surveyed during February and March, 1943.

These data also show the natural trend for cracking to increase with age. There is no indication that the type of joint had any effect on transverse cracking. Although the data for one age may favor one type of joint slightly, the data for another age are favorable to another type of joint. Generally speaking, the amount of cracking was the same for all joints. Perhaps the most significant aspect of the data shown in Table 55 is the high frequency of broken panels over culverts, which indicates clearly that there are features in the design of pavement slabs or the culverts under them, or both, which tend to cause cracking of the pavement slab. This defect appears to be sufficiently serious to warrant a study of the problem with the view of avoiding in some way the occurrence of these cracks. If this is not practicable, perhaps the installation of a construction joint with bonded tie bars to keep the crack closed and free from infiltration would be an improvement over present practice.

It is evident from this and previous investigations that joints spaced at 30-ft. intervals are not effective in preventing the formation of transverse cracks in pavements. In fact, it is probable that the rate of cracking on such pavements is greater at early ages than on unjointed pavements or pavements with joints at relatively large intervals, because the stresses due to restrained warping, which apparently are a primary contributing factor in the formation of transverse cracks, are of serious magnitude in 30-ft. slabs as well as in longer slabs. Thus, there is as much chance of a crack developing in a 30-ft. slab as in a longer slab. It is very probable that eventually all of the panels between the metal and premolded joints at close spacings will have cracked at least once, and, in the final analysis, the average length of unbroken slab will be approximately the same in jointed and unjointed pavements.

The data on cracking of pavement with 4-in. open joints agree substantially with those from previous surveys. It is indicated that at an age well within the anticipated service life, pavements with joints at close intervals may have as many transverse cracks as those with 4-in. open joints, and that they will present an equally serious maintenance problem.

Although corner failures have not reached serious proportions on any of the pavements covered by this survey, it seems reasonable to expect that the pavements with metal joints at 30-ft. intervals will be very susceptible to exterior corner failures, because there is no provision in these pavements for load transfer at transverse cracks. For this same reason, it is believed that "pumping" may become a serious problem as these pavements become older, due to the lack of mutual support between adjacent slabs.

V. Armington Experimental Road

16. *General.* — During the course of its investigation of joints in concrete pavements, the University committee proposed the construction of an experimental pavement for the purpose of providing: (1) an opportunity for manufacturers to demonstrate their products; (2) a means for observing the installation procedure and problems of various types of joints; (3) a means for making measurements of joint and slab movements; and (4) a concentrated grouping of various types of joints where their behavior could be studied over an extended period of time under the same conditions.

The plan was approved by the Division of Highways, and the project was included as a part of a regular paving contract on a road near the village of Armington about 25 miles southwest of Bloomington. The road is a stub running from Route 119 north for a distance of two miles and thence west into Armington. It was officially designated as Route 119-A, Section 118, Tazewell and Logan Counties, Regular Federal Aid Project No. 220-A.

This road was chosen partly because its location made it readily accessible for inspection by members of the University committee and engineers from the Division of Highways, but principally because of a combination of conditions which made it particularly desirable for an investigation of this nature. It was anticipated that the traffic using the road would be light, and it was believed that a truer picture of the actual behavior of joints due to expansion and contraction could be obtained if not complicated by heavy traffic. Furthermore, the road was desirable because of the uniform subgrade, flat grades, and minimum of cut and fill which it provided.

Data from the Bureau of Highway Research of the Division of Highways indicated that the traffic in 1940 averaged about 400 vehicles per day. Of this number 84 per cent were passenger cars and 16 per cent were trucks. Only 2 per cent of all vehicles were of the heaviest type permitted in Illinois (Class X), and observations showed that very few of these were fully loaded. Medium trucks carrying farm products usually provided the heaviest loads. It is thus seen that traffic on this road is very light, both in number and weight of vehicles. This should be kept in mind in studying the results of this investigation.

The experimental project on the road was divided into two parts, totaling 7,410 ft., or 1.403 miles, in order to take advantage of favorable topography. The first starts near the north end of the section at Sta. 2 + 00 and extends south a distance of 3,675 ft. to Sta. 38 + 75.

The other starts at Sta. 68 + 00 and extends south a distance of 3,735 ft. to Sta. 105 + 35. The general layout is shown in Fig. 92 (see pocket attached to inside back cover).

All arrangements for obtaining joints and load transmission devices for installation in the experimental project were made by the University committee, interested manufacturers being invited to participate by contributing a representative number of their products.

The material contained herein is essentially a progress report of the experimental project, covering the three-year period ending August 22, 1941, when the last detailed examination was made. It includes discussions of construction practices and problems, and observations made during installation; analysis of measurements made of horizontal and vertical movements of individual panels of pavement between joints; a study of seasonal movements and permanent closure of joints; and the results of visual inspections to determine the condition of the joints and such conditions of the pavement as were influenced by the joints and load transmission devices. Partial examinations were made after August 22, 1941, and where such results are given herein, the date is noted.

17. *Construction.* — The pavement was constructed as a regular contract under the direction of the Division of Highways. It was originally planned that the pavement would be built in the fall of 1937, but unexpected conditions compelled postponement of the work until the spring of 1938. Work was started on the north experimental part of the section on May 26, 1938, and finished June 3, 1938. The south part was started June 8, 1938, and finished June 16, 1938. The pavement was of the standard 9 in.-6½ in.-9 in. cross section, 20 ft. wide. The road was opened to traffic early in July, 1938.

The materials used in the construction of the pavement were supplied from the following sources:

Coarse aggregate (gravel) — Materials Service Corporation, Lockport, Ill.

Fine aggregate (sand) — Springfield-Pekin Sand and Gravel Company, Pekin, Ill.

Cement — Marquette, LaSalle, Ill. Lone Star, Greencastle, Ind.

In general, the construction procedure followed the usual practice prescribed by the Division of Highways in the Standard Specifications for Road and Bridge Construction adopted July 1, 1936, and the plans and special provisions effective at the time of construction. The various joints and load transmission devices were installed by the regular construction crew under the direction of representatives of the respective manufacturers.

At the first and last joint in each experimental group, the identification mark assigned to that group, the station number, and the date of installation were permanently imprinted in the surface of the concrete. The date was also placed at the beginning and end of each day's run and station numbers were imprinted at intervals, in accordance with usual practice in Illinois. These marks were imprinted with characters 5 in. high to a depth of about ½ in. They proved very helpful to observers in identifying the various groups during subsequent observations.

The air temperature during construction of the experimental sections ranged between 64 and 82 deg. F. at 8:00 a.m., 70 and 88 deg. F. at noon, and 75 and 90 deg. F. at 4:00 p.m. The average daily temperature was 78 deg. F.

The layout of the experimental portions of the road is shown in Fig. 92, which gives the type, location, and spacing of each joint and type of load transmission device used. As will be seen, the joints were installed in groups; that is, all the joints furnished by a manufacturer were, in general, placed consecutively. Usually there were five expansion joints and, when they were used, ten contraction joints. Various joint spacings were used in order to study the effect of this factor. From Sta. 0 + 00 to Sta. 2 + 00, from Sta. 38 + 75 to Sta. 68 + 00, and from Sta. 105 + 35 to the south end of the section, the contractor furnished J-5 metal expansion and contraction joints with L-1 load transmission devices as a regular part of his contract.

It is not necessary to describe the installation procedure for each brand of joint and load transmission device installed, but it is of interest to summarize the problems which were encountered in installing the several general types of joints and to relate the difficulties with some specific joints of each type.

(a) Metal Joints

The metal joints were so similar in general character that to a considerable degree they all presented the same installation problems. Because of their weight and lateral flexibility, three and sometimes four men were required to carry them into place. In some cases the shape of the copper seal, the position of the anchor pins, and the nature of the load transmission device were such that it was necessary to take extra precautions in placing and vibrating the concrete against the joints in order to prevent honeycomb.

The L-4 load transmission device had curved legs which rested on the subgrade to provide greater stability. In order to demonstrate this stability, the finishing machine was run over one of the J-6 joints containing this device, without raising the screeds. The joint was not displaced, but pieces of aggregate under the screeds bent and damaged the copper seal.

The J-2 joints were particularly heavy and extremely awkward to handle. Also, the heavy temporary anchor pins were hard to pull, causing considerable disturbance of the concrete adjacent to the joint which had to be corrected in the finishing process.

(b) Bituminous Premolded Fiber Joints

These joints were lighter in weight than the metal joints, but were so flexible that three men were required to place them. Most of the special installation problems, however, arose principally from the load transmission devices.

The L-14 load transmission devices were shipped separately and had to be assembled on the joint at the site of the work. The holes punched in the fiber joint material for the specially shaped dowels did not correspond to the dowel spacing of the load transmission system assembly, and many of them had to be recut. While an attempt was made to plug the unused holes with joint material, it is believed this was not properly done in all cases, and fins of concrete may extend across the joints at these places. However, no trouble from this cause is apparent.

As discussed in the description of the J-10 joint, one flange of the copper seal on this joint was very close to the surface of the pavement and quite short. Knowing this, special care was exercised to see that the concrete was properly placed and compacted around this flange. In spite of these precautions, several places were found where the short flange was loose in the newly hardened concrete, proving that such a short flange is ineffective and undesirable.

A feature peculiar to the J-8 joint was the wire "cage" for supporting the dowels. This joint was not in regular production, hence the cages for the experimental joints, being handmade, were rather crude and inaccurate and did not hold the dowels firmly in place or in proper alignment. Furthermore, it was observed that the cages obstructed the flow of the concrete during placing, and it is thought that honeycomb may have resulted. This would be particularly serious because the dowels, not having support from the concrete at the face of the joint, would be largely ineffective in transferring load from one slab to another.

(c) Bituminous Premolded and Rubber Joints

These joints were the conventional asphalt, felt and mineral filler type, and those made from sponge rubber. Because of the great flexibility of these materials, some method of supporting the joints during installation was required. Unfortunately the equipment on hand did not fit all of the different joints; consequently some of them were not so well placed as would be expected on jobs where proper equipment was available.

Keeping the load transmission devices in proper alignment was another problem with these joints. This was particularly true of the short dowel unit, such as the L-1 device, which depends for alignment on its limited bearing against the joint material. The area of bearing on the soft and flexible material was entirely too small to furnish adequate support to the load transmission device. On the other hand, the large bearing plates on the L-10 and the L-11 devices held those devices securely in line. It was necessary to place supports at both ends of long dowels to hold them in good alignment.

(d) Special Types

In installing the "non-extruding" joint, it was observed that in most cases the side plates were forced away from the joint material, leaving a space through which dirt could enter the extrusion chambers.

Difficulty was encountered in forming the space and supporting the dowels for the joint with the molded rubber seal, but the rubber seal had nothing to do with this trouble.

(e) Dummy Joints

These joints were formed by driving a T-iron of proper size into the concrete to form a groove, soon after the finishing machine had passed. T-irons of 1, 1½, 2, and 2½ in. were used in this experiment. The 1-in. size was too light to handle and did not drive well, while the 2- and 2½-in. sizes were too large. From the standpoint of operation, the 1½-in. size appeared to be the best. Where a premolded filler was to be used, it was inserted in the groove, and the surface of the concrete disturbed by the grooving operation was finished and edged along the filler. When a poured filler was to be used in the groove, a strip of weatherboard was inserted in the groove to aid the final finishing operation and was then removed. The first joints formed by the latter method were not well finished, because the need for wood strips was overlooked and the concrete had begun to set before weatherboards were obtained.

There appeared to be some installation advantage in using premolded strips in the grooves. It was found that where this material was used, the grooves were all full depth. Where wood strips were used, the grooves varied considerably in depth, probably because the strips were withdrawn too early and mortar flowed into the bottom of the groove.

(f) Wood Joints

These consisted of 1-in. cypress boards, equipped with L-1 load transmission devices. Because of their rigidity, they were by far the easiest to install of any of the joints used on the experimental road. The firmness of the wood held the load transmission devices in excellent alignment. Their stiffness and lightness made them very easy to handle, and two men could install them without difficulty. It was easy to drive the supporting stakes tightly against the joints, leaving no pockets for the formation of honeycomb.

18. *Field Observations and Measurements.* — Examinations were made from time to time by members of the University committee and engineers from the Division of Highways to observe the condition of the pavement, joints, and load transmission devices. On some of those occasions, measurements were made to determine the movements which were occurring at joints at selected locations, in general at two expansion joints and one contraction joint in each group. Measurements included opening and closing of joints, horizontal movement of entire panels between joints, and vertical movements of slab edges adjacent to joints.

The opening and closing movements were made between gage plugs consisting of copper roofing nails set in pairs straddling the joint with their heads flush with the pavement surface. Four pairs were installed at expansion joints, one pair 6 in. from each edge and a pair 6 in. on each side of the longitudinal center joint. At contraction joints only the two pairs at the edges were installed. As soon as the concrete had set sufficiently hard, center punch marks were placed in the nail heads at a basic gage distance of 8 in., except at the 4-in. open joints, where the gage distance was made 11 in.

Horizontal and vertical slab movements were taken from the gage plugs referred to bench marks placed near the right-of-way lines for that purpose. These consisted of concrete cylinders 6 in. in diameter and 5 ft. long, cast in place, with a steel rod extending vertically through the concrete into the soil below. These were also set in pairs on the extended centerline of the joint, one on each side of the pavement, with punch marks placed in the top of the steel rods in such a manner that a line from the punch mark on one bench mark to that on the corresponding bench mark on the other side of the pavement was parallel to the centerline of the joint, and coincided with the line between the outside pair of gage plugs on that side of the joint.

Horizontal movements of slab panels were taken by setting a transit over one bench mark, orienting it on the one directly opposite it on the other side of the pavement, and measuring the position of the gage plugs with respect to this line.

As soon as possible after the pavement was finished, profiles were taken along each edge of the pavement, elevations being taken at the gage plugs at the end of the joint and at the midpoint of each slab panel. Profiles were also taken along the longitudinal center joint by taking elevations at the gage plugs set on each side of the transverse joints. The readings were made with an engineer's level to 0.005 ft., and carefully checked by reference to both bench marks in each pair.

While neither of these methods provides extreme accuracy, significant results were obtained, and it appears that the data are sufficiently reliable to indicate the general behavior of the panels under observation.

Initial readings were taken on the north section on June 9, 1938, and on the south section on June 20 and 21, 1938. Up to and including August 13, 1941, nine additional sets of readings had been taken to determine the movements at joints and six to determine horizontal and vertical movements of slabs. Readings to determine the width of the joint openings were also made in July, 1943, and November, 1943. Detailed examinations of the condition of the joints and pavements were made on several occasions, the last on August 12 and 13, 1941, and a number of partial surveys were made at various times both before and since that date. It is planned to continue making measurements and observations as long as they yield valuable information.

The data which have been accumulated are too voluminous to report in detail in this bulletin. For that reason only typical examples showing trends in movements of joints and panels are given. The condition of the joints and pavement is indicated by summaries of the results of the latest complete condition survey made during August, 1941, supplemented by discussion of the results of partial surveys made later. Discussions of the results of measurements and observations follow.

(a) Movements at Expansion Joints

Typical examples of horizontal movements at expansion joints are shown for selected joints in Figs. 93 to 97, inclusive. In these graphs, changes from the initial gage width of the joints are plotted as a function of age for the 11 times that readings were taken.

Included in Fig. 93 are graphs for the metal joints which had a nominal opening of 1 in. Figure 94 includes the 1-in. joints with premolded fillers. The ½-in. metal and premolded joints are covered by Figs. 95 and 96. Movements at 4-in. open joints are shown in Fig. 97.

As would be expected, the graphs show that the joints respond to temperature changes; as the temperature falls the joints open, and as it rises they close. It was noted, however, that the joints did not

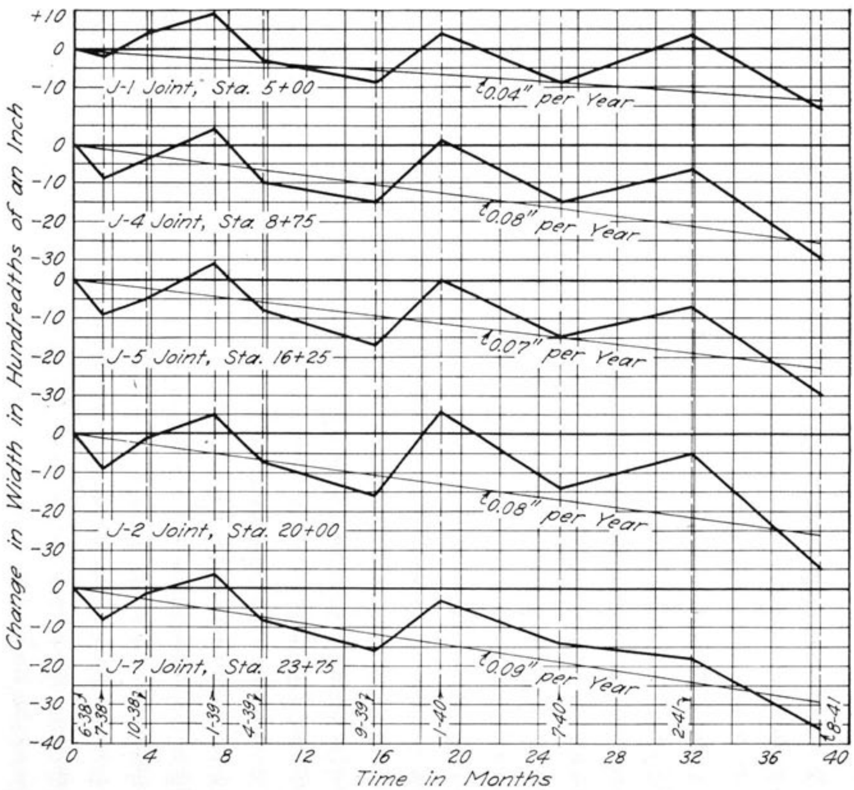


FIG. 93. CHANGE IN WIDTH OF 1-IN. METAL EXPANSION JOINTS INSTALLED AT 75-FT. INTERVALS WITH TWO INTERVENING CONTRACTION JOINTS FORMING 25-FT. PANELS

open and close uniform amounts, the movement being more at some joints than at others. This variation in joint behavior is significant, as it may have an important bearing on the formation of cracks, blowups, and other failures. The reason for unequal movements at joints was not ascertained, but it probably was due principally to differences in internal resistance within the joints caused by the load transmission devices and the presence or absence of joint filler. This is indicated by the fact that most of the premolded type joints showed smaller movements than the metal joints, which were designed to provide a substantially free expansion space.

Figures 93 to 97 also show that many of the expansion joints have developed an appreciable amount of permanent closure; that is, the joint opening is becoming smaller as the age of the pavement increases. In these cases, the approximate average rate of closure for the three-year period was estimated by plotting by inspection an average straight line for the various points on the curve.

Here also the type of joint appears to have an effect; the joints having free expansion spaces, in general, show more permanent closure than the premolded joints, which offer more resistance to closing. It

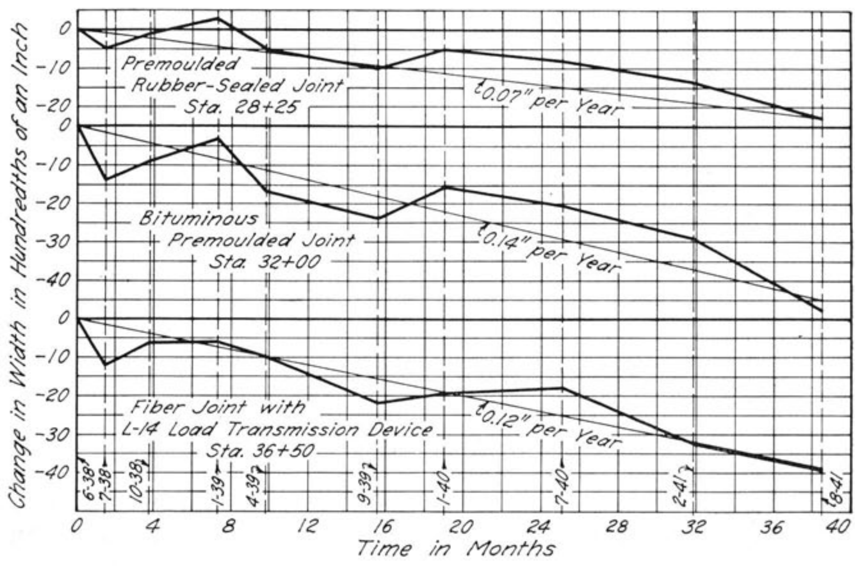


FIG. 94. CHANGE IN WIDTH OF 1-IN. PREFORMED EXPANSION JOINTS INSTALLED AT 75-FT. INTERVALS WITH TWO INTERVENING CONTRACTION JOINTS FORMING 25-FT. PANELS

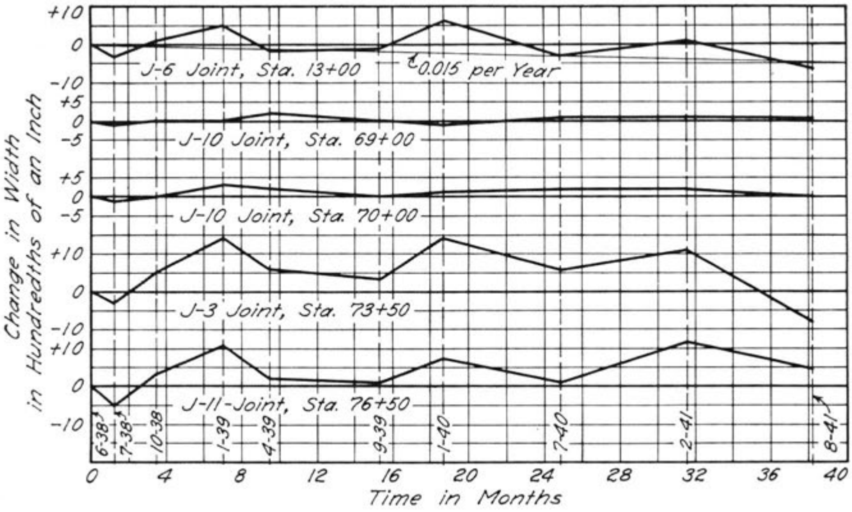


FIG. 95. CHANGE IN WIDTH OF ½-IN. EXPANSION JOINTS INSTALLED AT 25-FT. INTERVALS

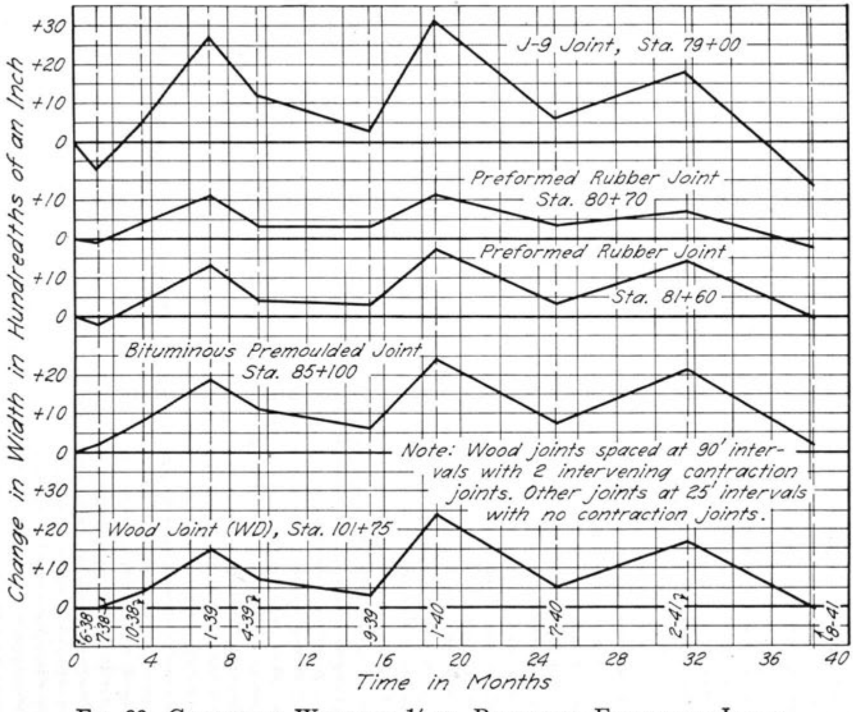


FIG. 96. CHANGE IN WIDTH OF ½-IN. PREFORMED EXPANSION JOINTS INSTALLED AT 25 AND 30-FT. INTERVALS

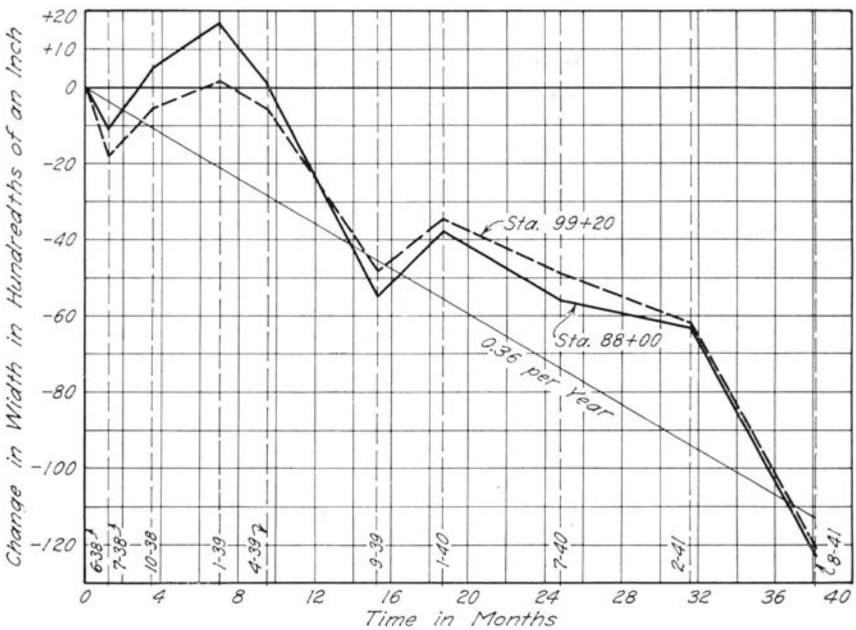


FIG. 97. CHANGE IN WIDTH OF 4-IN. OPEN JOINT

further appears that the amount of permanent closure is affected by the presence of contraction joints, the greatest closure occurring at expansion joints installed in conjunction with two contraction joints. The reason for this is discussed in Sections 2 and 15(d) (pages 18 and 159), where it is pointed out that pavements are subject to growth due to infiltration of dirt into transverse cracks and contraction joints.

The rate of closure for selected expansion joints for both the three-year period and the five-year period is given in Table 56. The final readings for the three-year period were not all made at times when the temperature was approximately the same as when the initial readings were taken, and it was necessary to estimate the rate of closure by inspection. The rate of closure for the five-year period was determined from the total closure measured in August, 1943, since the measurements were taken when the pavement was at practically the same temperature as when the initial readings were taken in June, 1938.

It appears significant that practically every joint measured in the north section (Sta. 2 + 00 to Sta. 73 + 75) shows definite permanent closure. In this section, with the exception of a length of 150 ft., there are contraction joints between expansion joints, and all of the expansion joints had free expansion space or an easily compressible filler. The two J-5 joints at Sta. 100 + 02.5 and Sta. 100 + 85 also show a considerable permanent closure.

By way of contrast, in the south section, where there are no intervening contraction joints and the joints are narrower and contain fillers which offer some resistance to movement, practically no joint showed a determinable rate of permanent closure.

The greatest total permanent closure occurred at the 4-in. open joints at Sta. 88 + 00 and Sta. 99 + 20. This is as might be expected,

Table 56
Average Yearly Rates of Closure of Expansion Joints
(Armington Experimental Road)

Station	Joint	Type	Joint Spacing ft.		Three-Year Yearly rate		Five-Year Yearly rate	
			Exp.	Con.	In. per joint	In. per 100 ft.	In. per joint	In. per 100 ft.
2+75	J-1	1-in. metal	75	25	0.03	0.040	0.032	0.043
5+00	J-1	1-in. metal	75	25	0.04	0.053	0.044	0.059
7+25	J-4	1-in. metal	75	25	0.08	0.107	0.058	0.077
8+75	J-4	1-in. metal	75	25	0.08	0.107	0.050	0.067
10+25	J-6	1-in. metal	75	25	0.06	0.080	0.040	0.053
14+00	J-5	1-in. metal	75	25	0.06	0.080	0.050	0.067
16+25	J-5	1-in. metal	75	25	0.07	0.093	0.062	0.083
18+50	J-2	1-in. metal	75	25	0.06	0.080	0.054	0.072
20+50	J-2	1-in. metal	75	25	0.08	0.107	0.068	0.091
22+25	J-7	1-in. metal	75	25	0.10	0.133	0.084	0.112
23+75	J-7	1-in. metal	75	25	0.09	0.120	0.078	0.104
26+00	J-3	1-in. metal	75	25	0.09	0.120	0.096	0.128
100+02.5	J-5	1-in. metal	82.5	27.5	0.09	0.109	0.064	0.077
100+85	J-5	1-in. metal	82.5	27.5	0.11	0.133	0.086	0.104
Average					0.074	0.097	0.062	0.081
26+75	GR	1-in: open	75	25	0.08	0.107	0.058	0.770
27+50	GR	1-in. open	75	25	0.08	0.107	0.084	0.112
28+25	GR	1-in. soft filler	75	25	0.07	0.093	0.066	0.088
31+25	PC	1-in. soft filler	75	25	0.12	0.160	0.114	0.152
32+00	PC	1-in. soft filler	75	25	0.14	0.187	0.136	0.181
35+75	L-14	1-in. soft filler	75	25	0.15	0.200	0.096	0.128
36+50	L-14	1-in. soft filler	75	25	0.12	0.160	0.094	0.125
Average					0.108	0.144	0.093	0.124
12+50	J-6	½-in. metal	25	None	0.010	0.040	0.012	0.048
13+00	J-6	½-in. metal	25	None	0.015	0.060	0.014	0.056
Average					0.013	0.050	0.013	0.052
37+85	L-14	½-in. soft filler	30	15	0.005	0.017	0.008	0.027
38+15	L-14	½-in. soft filler	30	15	0.070	0.233	0.068	0.427
Average					0.038	0.125	0.038	0.227
88+00		4-in. bit.	1,120	Var.	0.46	0.041	0.45	0.040
99+20		4-in. bit.	1,120	Var.	0.46	0.041	0.51	0.046
Average					0.46	0.041	0.48	0.043

Note: The three-year rates were estimated by means of rate of closure lines made by inspection on the graphs for that period, since times and temperatures were irregular.
The five-year rates were determined from the total closures in 1943, since the measurements were taken when the pavement was at practically the same temperatures as when the plugs were installed.

because of the relatively great length of pavement (1,120 ft.) between these joints. The rate per 100 ft., however, is only about 40 per cent of that for metal joints.

Table 56 shows the five-year average rate of closure for 1-in. metal expansion joints, spaced 75 and 82.5 ft., to be 0.06 in. per year per joint; and for 4-in. open joints, spaced 1,120 ft., about 0.5 in. per year per joint. These figures indicate that metal joints might be expected to close completely in 10 to 12 years, and 4-in. open joints in eight to 10 years. Data from the field investigations discussed earlier in this bulletin indicate that complete closure of metal joints could be expected in eight years and of 4-in. open joints in about five years. Because joints do not close equal amounts, it is reasonable to expect that some joints may close completely much earlier than the indicated average time.

(b) Movements at Contraction Joints

Typical examples of the movement in contraction joints are shown in Figs. 98 to 100, inclusive. Movements in seven representative metal contraction joints are shown in Fig. 98. Figures 99 and 100 give

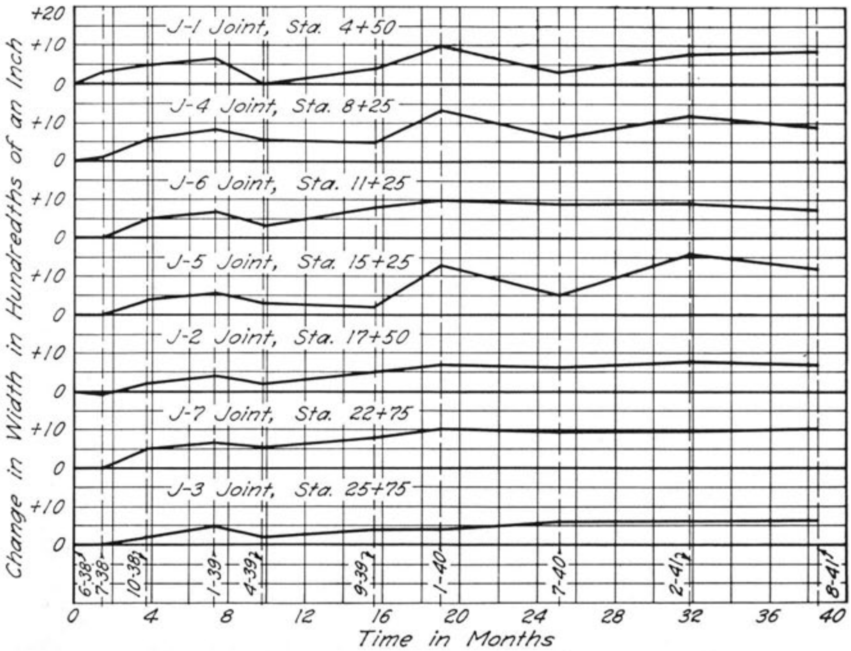


FIG. 98. CHANGE IN WIDTH OF METAL CONTRACTION JOINTS

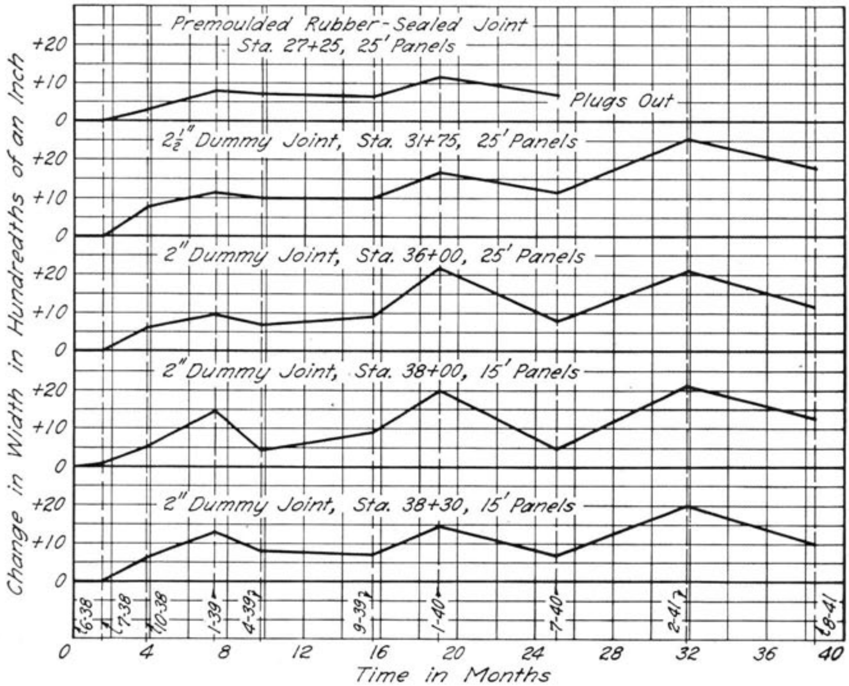


FIG. 99. CHANGE IN WIDTH OF DUMMY CONTRACTION JOINTS INSTALLED AT 15 AND 25-FT. INTERVALS

similar data for 12 dummy contraction joints. The five joints covered by Fig. 99 were installed in conjunction with 1-in. expansion joints, two dummy contraction joints between each pair of expansion joints. The seven dummy joints included in Fig. 100 were installed without intervening expansion joints.

It will be noted that all the contraction joints exhibit a general progressive tendency to open up permanently, in contrast to the opposite tendency of expansion joints, further illustrating the fact that pavement slabs act more or less collectively during expansion and individually during contraction. Soil sifting into the open contraction joints during each successive period of contraction causes progressive opening of contraction joints and progressive closing of expansion joints.

Generally speaking, there were somewhat greater movements in the dummy contraction joints than in the metal ones. This may have been due to resistance set up by the load transmission devices with which the metal joints were equipped; the dummy joints contained no such devices. It is possible that the copper seals were somewhat effective in keeping dirt out of the metal joints, thereby reducing the permanent opening in this type of joint.

As in the case of expansion joints, the movements in contraction joints also were not uniform, those at some joints being greater than at others. The unusually large movement in the dummy joint at Sta. 88 + 35 was probably influenced by the large permanent closure of

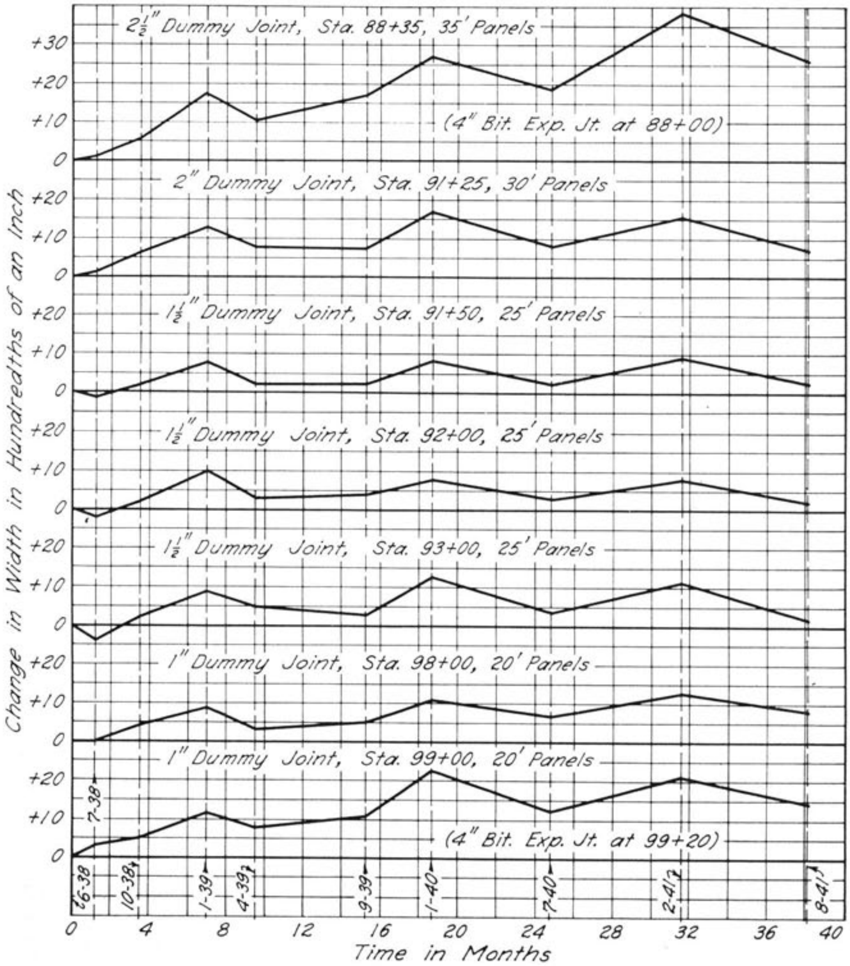


FIG. 100. CHANGE IN WIDTH OF DUMMY CONTRACTION JOINTS INSTALLED AT 20, 25, 30, AND 35-FT. INTERVALS WITH NO INTERVENING EXPANSION JOINTS

the 4-in. open joint at Sta. 88 + 00, only 35 ft. away. Measurements made on August 16, 1944, indicated that the opening of dummy contraction joints immediately adjacent to the 4-in. open joints at Sta. 99 + 20 and Sta. 88 + 00 were from two to three times greater than the opening of the dummy joints farther away from the expansion joints. This effect was apparent only 100 ft. on each side of the expansion joints, the openings in the middle part of the section being approximately equal for all joints.

There are indications that some of the dummy joints opened more than would be expected from contraction only, while adjacent joints opened less than the anticipated amount. As observations showed that at times the joint openings were full of water, this possibly might be explained by water freezing in the joint and exerting pressure tending to open it. Since the water in the narrower joints would freeze first, the volume of water being smaller than in wider joints, pressure in the former joints might move the adjacent slabs toward wider joints in which water had not frozen. This may partially explain the permanent closure of expansion joints and the opening of intervening contraction joints. The possible effect of ice pressures in joints has been given very little consideration and study.

(c) Horizontal Movements of Slabs

The horizontal movement of slabs with respect to bench marks is plotted in Figs. 101 to 106, inclusive. In Figs. 101 to 103, inclusive, the movements of the end of the slabs on one side of selected joints are shown, the movement at each end of the joint being plotted separately. For the most part, the movements of the two corners of each slab were in reasonably good agreement, the difference being no greater than the experimental error which might be expected with the method of measurement used. Only the readings made in August, 1941, at Sta. 95 + 20 (Fig. 106) indicated any appreciable torsional displacement of the slab. In this case the east corner of the south slab was 0.05 in. north and the west corner 0.17 in. south of their original positions, a difference in relative positions of about ¼ in.

It appears that permanent movements of some of the slabs occurred over the three-year period during which readings are available. For example, Fig. 101 shows that in August, 1941, the slab on the north side of the joint at Sta. 36 + 00 was approximately 0.25 in. north of its original position determined in June, 1938.

In August, 1941, the slab on the south side of the joint at Sta. 70 + 00 was approximately 0.15 in. north of its original position, also determined in June, 1938 (Fig. 102). It is interesting to note that the movement at Sta. 36 + 00 was upgrade, while the movement at Sta. 70 + 00 was downgrade.

The greatest movement recorded for any slab was on the south side of the 4-in. open joint at Sta. 88 + 00. Figure 103 shows an apparent permanent movement of approximately 1.27 in. northward, which is upgrade. A possible explanation is that the point measured is the north end of a section of pavement 1,120 ft. long, which contains dummy contraction joints at 20- to 35-ft. spacings and has no provision for taking up expansion and permanent movements except the 4-in. open expansion joints at each end. When this fact is considered, it is reasonable to expect a large permanent movement at the 4-in. joints. It is unfortunate that similar measurements were not made at Sta. 99 +20, the other end of the 1,120-ft. section of pavement, between the two 4-in. joints.

The movement of the slab ends on each side of selected expansion joints and the shift of the joint centerline are shown in Figs. 104 and

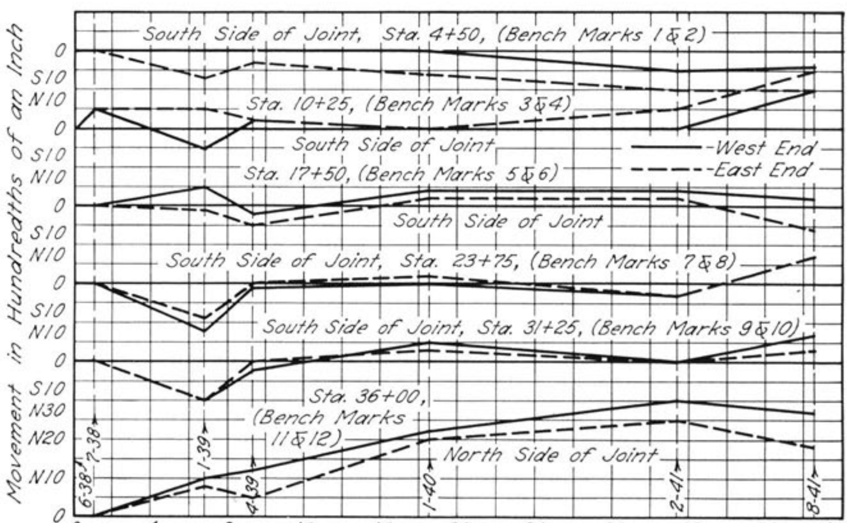


FIG. 101. HORIZONTAL MOVEMENT OF SLABS ADJACENT TO JOINTS WITH REFERENCE TO BENCH MARKS

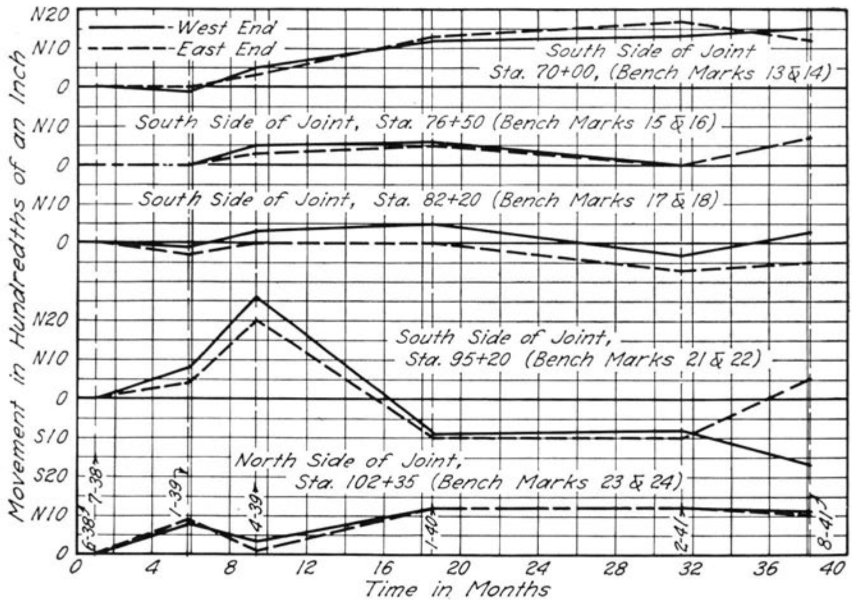


FIG. 102. HORIZONTAL MOVEMENT OF SLABS ADJACENT TO JOINTS WITH REFERENCE TO BENCH MARKS

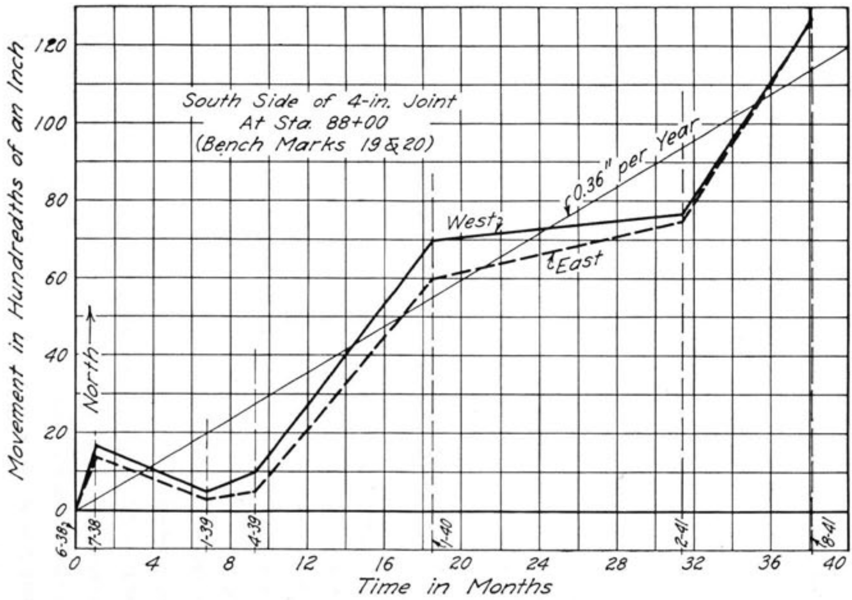


FIG. 103. HORIZONTAL MOVEMENT OF SLAB ADJACENT TO 4-IN. OPEN JOINT WITH REFERENCE TO BENCH MARKS

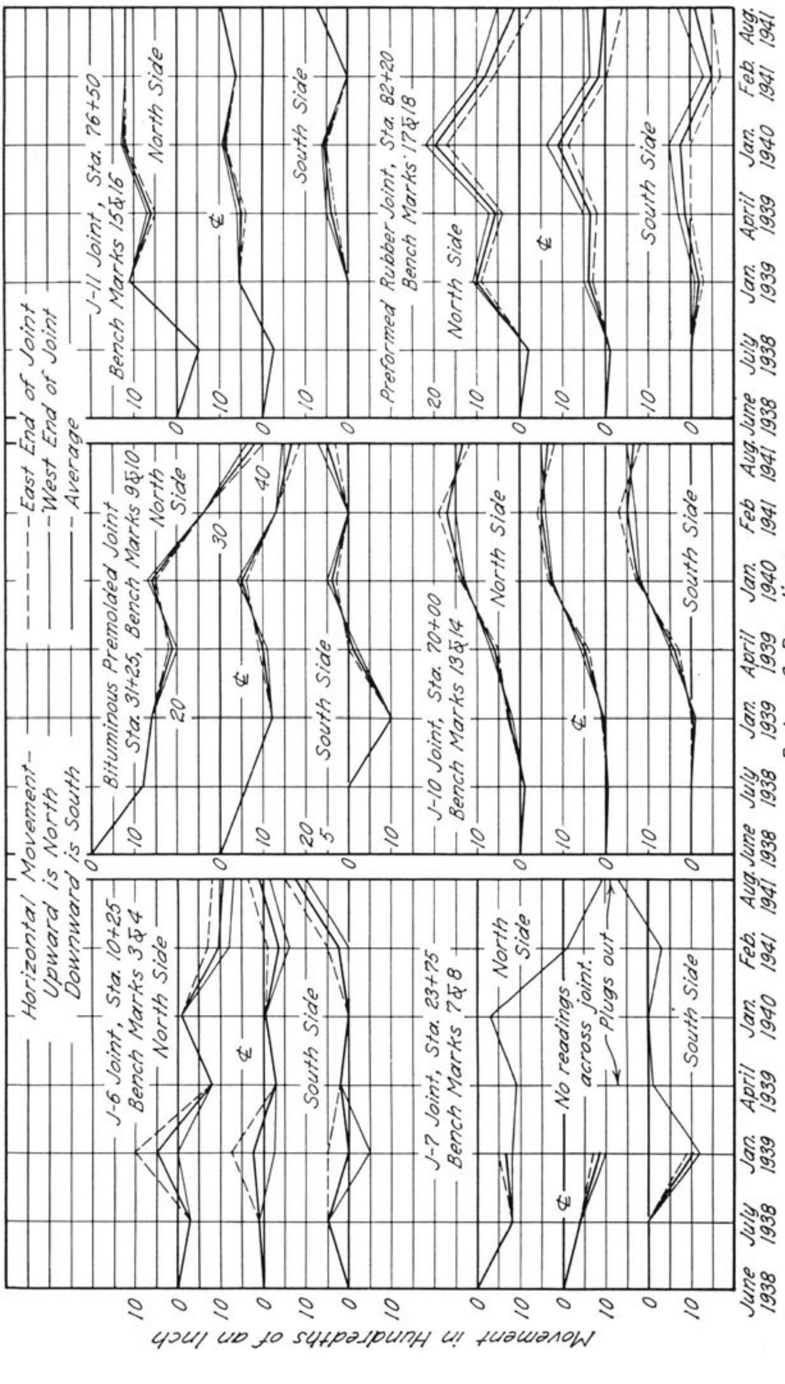


FIG. 104. HORIZONTAL MOVEMENTS OF SLAB ENDS ON EACH SIDE OF EXPANSION JOINTS AND SHIFT IN CENTERLINE OF JOINTS

105. Besides opening and closing with seasonal change in temperature, some of these joints shifted permanently from their original positions. This is apparent from the fact that the average centerline position in August, 1941, deviates considerably from the zero or original position. Examples of shift in position are the bituminous premolded joint at Sta. 31 + 25, the J-10 joint at Sta. 70 + 00, shown in Fig. 104, and the 4-in. open joint, shown in Fig. 105. The most striking example

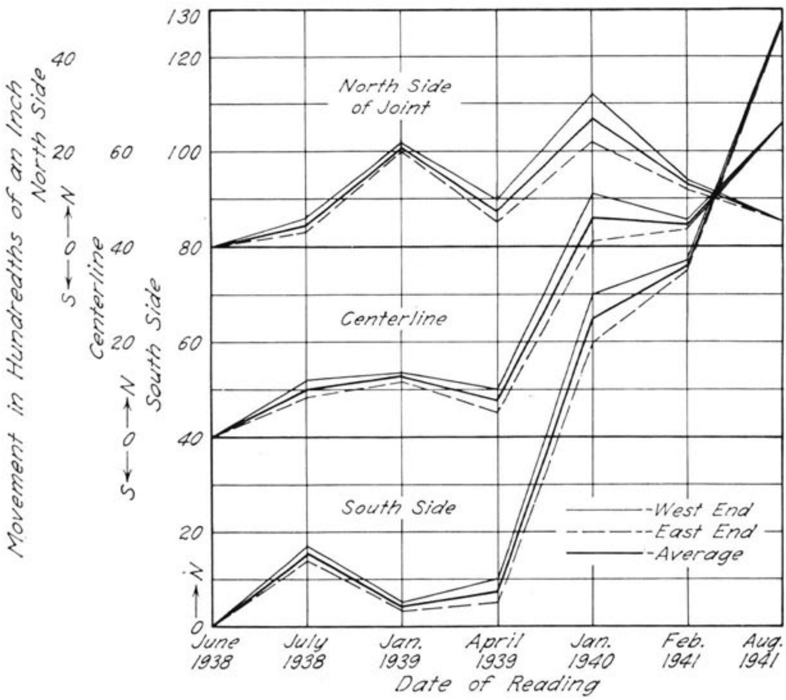


FIG. 105. HORIZONTAL MOVEMENTS OF SLAB ENDS ON EACH SIDE OF 4-IN. OPEN JOINT AT STA. 88 + 00 AND SHIFT IN CENTERLINE OF JOINT

is the latter, where the south side moved north a distance of 1.27 in., while the north side was moving 0.05 in. to the south, resulting in a permanent closure of 1.32 in. and a shift in the position of the joint centerline of 0.61 in. to the north.

Similar movements for two selected contraction joints are shown in Fig. 106. It is seen that the movements at these joints have been minor.

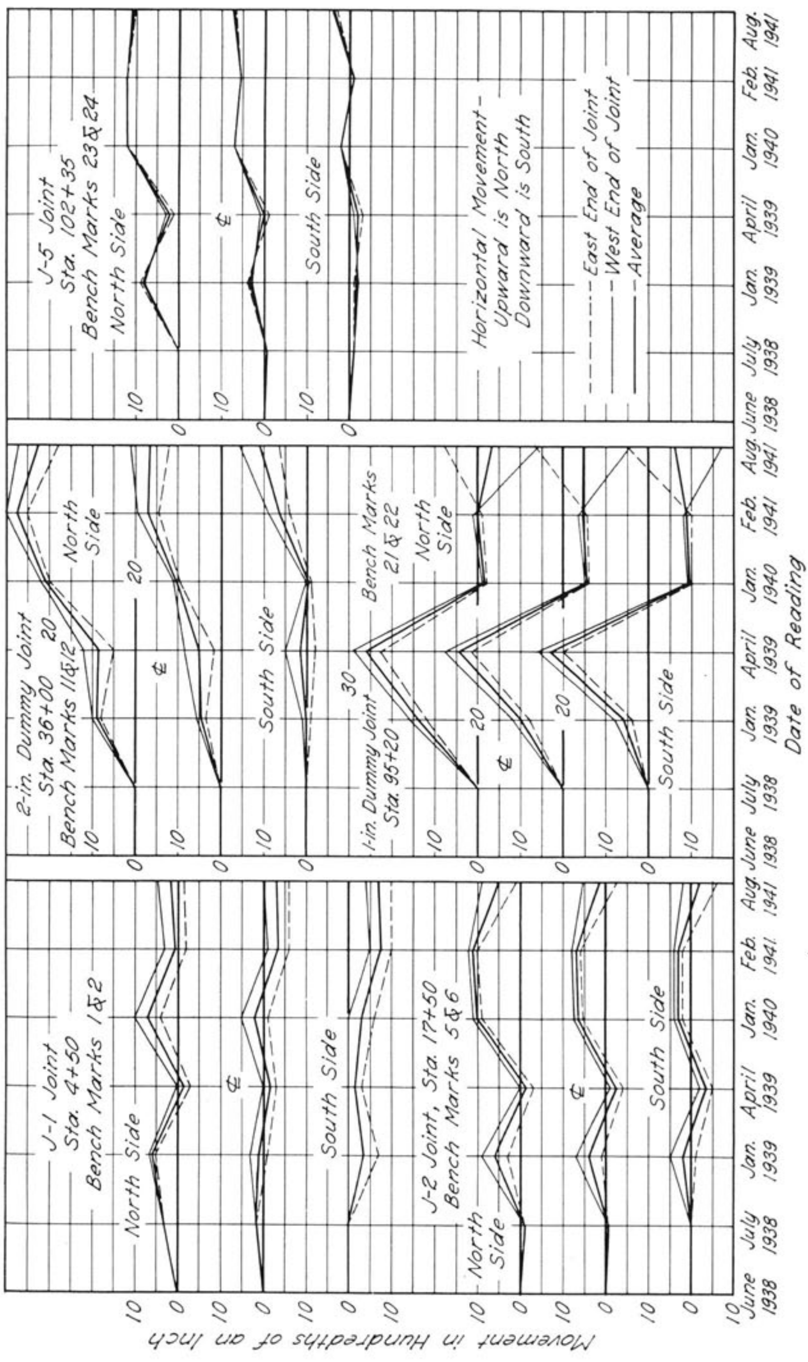


FIG. 106. HORIZONTAL MOVEMENTS OF SLAB ENDS ON EACH SIDE OF CONTRACTION JOINTS AND SHIFT IN CENTERLINE OF JOINTS

(d) Vertical Movements of Slabs

Profile readings were taken along each edge of the pavement as soon as the concrete had set sufficiently, readings being taken on both sides of each joint and at the midpoint of each panel. Elevations were also taken along the centerline of the pavement, where gage plugs were placed at joints. The readings, made with an engineer's level to 0.005 ft., were carefully checked from each pair of bench marks. While this method does not provide extreme accuracy, it is sufficiently accurate to indicate the general behavior of slabs with respect to vertical movements. Six sets of readings were taken at different seasons of the year between June, 1938, and August, 1941.

The data from these readings are too voluminous to present in tabular form in this bulletin. Complete profiles of the west edge of the pavement, plotted for the purpose of study, are not included because of their size. A discussion of the results of the study is of interest, however.

The general elevation of the pavement changed with the seasons, being higher in the winter and lower in the summer. The maximum movements recorded during the three-year period ranged from 0.05 to 0.1 ft. There were some indications of permanent heaving and settlement, the former being found principally in cuts or at transitions from cut to fill, and the latter on the higher fills; but none of these movements exceeded 1.0 in.

In most cases equal changes in elevation did not occur on each side of the joints, one slab almost invariably being higher than the other. There was no regularity in this action, the higher elevations occurring as often on one side of the joint as on the other. Nor was there any uniformity exhibited from season to season; that is, the difference in elevation between two sides of a joint did not remain constant, but changed from time to time and even reversed itself, the higher side sometimes becoming lower than the other side during the period between two readings. The differences in elevation across joints ranged from 0.005 to 0.025 ft. No attempt was made to correlate these data with respect to the types of load transmission devices used with the joints.

While the differences in elevation have not been large, it has been shown in an earlier discussion that relatively small changes in the contour of a pavement surface can change the riding quality of a pavement considerably. It has been observed that pavements, particularly those with joints at close intervals, become rougher during periods of cold weather, and it is entirely possible that differences in elevation of the magnitude found on the experimental sections may be a factor in the riding quality of a pavement.

(e) Condition of Copper Seals

As previous experience had shown that the copper seals on joints were susceptible to early cracking, the seals on the joints installed in the experimental road were observed closely for evidence of failure. A number of visual examinations, and one detailed examination in which the length of failures was measured, have been made.

The seals on all the metal air-chamber expansion joints were examined in detail on August 12 and 13, 1941. The bituminous caps and asphalt seals which had not already been displaced by traffic were removed, and the seals were examined closely for cracks and splits. Each failure was measured for length. The results of that survey are given in Table 57. Twenty-nine of 37 expansion joints in the section, or 78 per cent, showed definite and measurable failures.

Figures 107-130 afford further information.

The total length of splits in each joint is recorded in Table 57 in inches, and also in percentages of the length of the joint. Average values for each type of joint are also given. These data show definitely that the J-4, J-5, and J-6 joints had more extensive failures than other types of joints. These joints have the M-shaped or inverted U-shaped seal, which were found from laboratory tests and previous field investigations to be particularly susceptible in failure. On the other hand, however, the seals on the J-2, J-3, and J-7 joints installed in the experimental section, which were also of that shape, had a much lower percentage of failure, tending to indicate that the shape of the seal has no significance.

Considering the joints collectively, the total length of splits was about 1,662 in. out of a total length of seals of 8,880 in., or about 18.5 per cent. All of the seals were badly crimped by the expansion of the concrete and the action of traffic, and it was impossible to examine some of them thoroughly because of the tightness of the crimps and the remnants of asphalt caps which were embedded in the folds of the copper. The seals may have been split at some of these unobservable locations; therefore the values in Table 57 may be somewhat low. It was already apparent, at the end of three years of service on a pavement carrying very light traffic, that the copper seals had failed to such an extent as to establish definitely that seals of this type do not possess an adequate effective service life.

Table 57

Results of Copper Seal Survey

(Armington Experimental Road)

Expansion Joints—North Section—Sta. 2+00 to Sta. 26+00—August 12-13, 1941

Station	Type	Number of Joints	Number Failed	Percentage Failed	Number of Failures	Total Length of Failures, in.	Percentage Length Failed	Notes
2+75	J-1	..	..	....	3	15	6.3	Crimped almost tight at top.
3+50		..	..	....	2	4	1.7	Crimped to ⅛-in. rad. at top.
4+25		..	..	....	0	0	0	Crimped to 1/16-in. rad. at top.
5+00		..	..	....	8	44	18.3	Crimped over split in west lane.
5+75		..	..	....	4	16	6.7	Crimped to 1/16-in. rad. at top.
		5	4	80.0	17	79	6.6	
6+50	J-4	..	..	....	2	180	75.0	Crimped.
7+25		..	..	....	4	138	57.6	Crimped.
8+00		..	..	....	0	0	0	Crimped so tight that bottom could not be inspected. Probably failed at bottom. Crack 15 ft. north, open ¼-in.
8+75		..	..	....	5	132	55.0	Crimped.
9+50		..	..	....	8	194	80.8	Crimped.
		5	4	80.0	19	644	53.8	
10+25	J-6	..	..	....	7	69	28.8	Crimped.
11+00		..	..	....	5	49	20.4	Crimped.
11+75		..	..	....	4	22	9.6	Crimped.
12+00		..	..	....	0	0	0	Crimped.
12+25		..	..	....	1	12	5.0	10-in. spall 2½ ft. from east edge exposes edge of seal and admits water.
12+50		..	..	....	0	0	0	Crimped.
12+75		..	..	....	3	34	14.2	Crimped.
13+00		..	..	....	0	0	0	5-in. spall 3 ft. from east edge exposes edge of seal and admits water.
13+25		..	..	....	9	86	35.9	Crimped.
		9	6	66.7	29	272	22.6	
14+00	J-5	..	..	....	2	180	75.0	Crimped.
14+75		..	..	....	2	54	22.5	Crimped.
15+50		..	..	....	1	2	0.8	Crimped.
16+25		..	..	....	6	150	62.5	Crimped.
17+00		..	..	....	4	80	33.3	Crimped.
		5	5	100	15	466	38.9	
17+75	J-2	..	..	....	1	1	0.4	Failure starting—middle east lane.
18+50		..	..	....	1	3	1.2	Crimped.
19+25		..	..	....	5	28	11.7	Crimped.
20+00		..	..	....	4	15	6.3	Crimped.
20+75		..	..	....	0	0	0	Crimped tight. Difficult to inspect. Possibly failed.
		5	4	80.0	11	47	3.9	
21+50	J-7	..	..	....	1	7	2.9	Crimped.
22+25		..	..	....	1	2	0.8	Crimped.
23+00		..	..	....	12	74	30.8	Crimped.
23+75		..	..	....	0	0	0	Crimped tight. Possibly failed.
24+50		..	..	....	5	37	15.4	Crimped.
		5	4	80.0	19	120	10.0	
25+00	J-3	..	..	....	1	1	0.4	Old test punctures at 3 in. and 3 ft. from west edge. Split starting at latter. Hard to clean.
25+50		..	..	....	3	16	6.7	Hard to clean because of crimp. Closed to about ½-in.
26+00		..	..	....	0	0	0	Unable to remove asphalt from tight crimp, hence possible failures not observable.
		3	2	66.7	4	17	2.4	

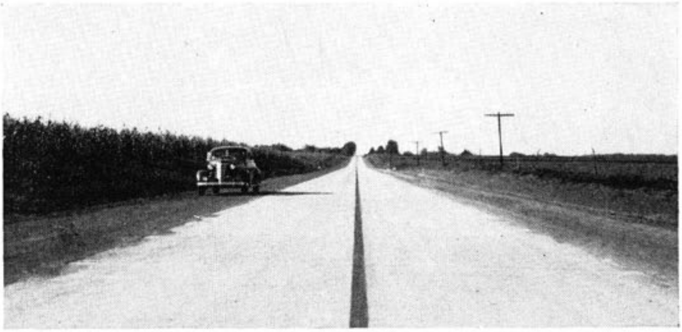
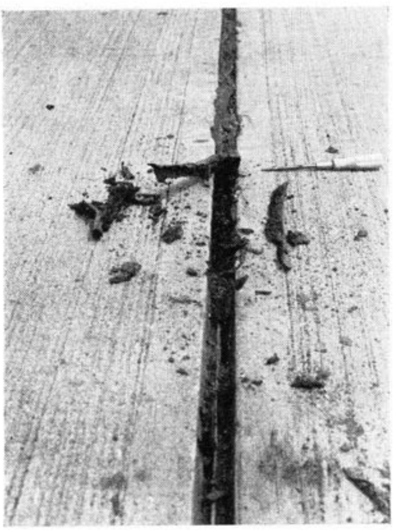


FIG. 107. VIEW OF SOUTH EXPERIMENTAL SECTION LOOKING NORTH FROM ABOUT STA. 100 + 00 (OCTOBER, 1938)

The copper seals on the metal contraction joints were not examined in detail, principally because the limited width of the joint made it impossible to expose the copper to thorough inspection. Such examinations as could be made indicated few, if any, failures at the end of three years. It is apparent that contraction joint seals have a considerably greater service life expectancy than expansion joint seals. This



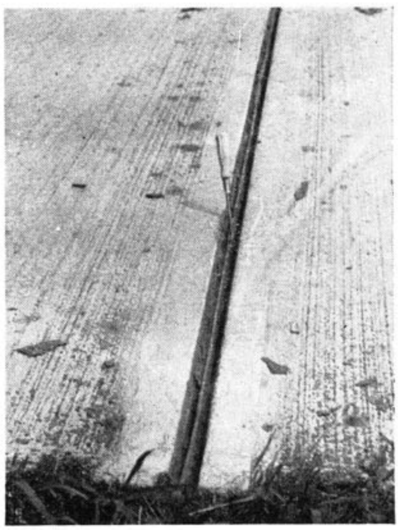


FIG. 108 (AT LEFT). J-1 EXPANSION JOINT AT STA. 5 + 75, WITH ASPHALT CAP LOOSENED BY DIRT AND PARTIALLY REMOVED, SHOWING COPPER SEAL SHARPLY CRIMPED (AUGUST 12, 1941)

FIG. 109. J-1 EXPANSION JOINT AT STA. 5 + 00 WITH ASPHALT CAP REMOVED, SHOWING COPPER SEAL CRIMPED AND SPLIT (AUGUST 12, 1941)

FIG. 110. J-4 EXPANSION JOINT AT STA. 9 + 50, SHOWING COPPER SEAL BEFORE ASPHALT CAP WAS POURED (JUNE, 1938)

FIG. 111. J-4 EXPANSION JOINT AT STA. 9 + 50 WITH ASPHALT CAP PARTIALLY REMOVED, SHOWING COPPER SEAL CRIMPED AND SPLIT (AUGUST 25, 1941)

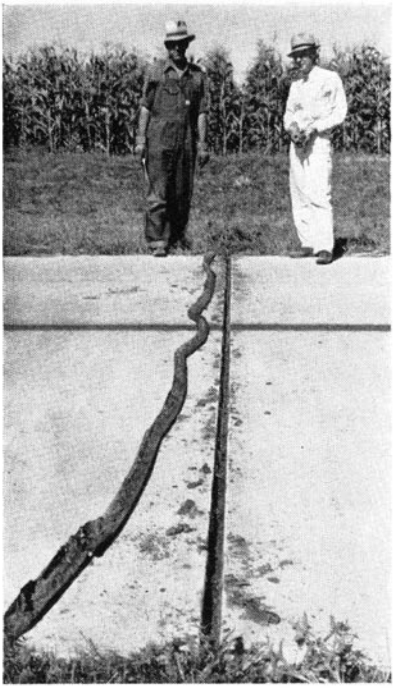


FIG. 112. J-6 EXPANSION JOINT AT STA. 13 + 00 SHOWING HOW POURED ASPHALT CAP, WHICH DID NOT ADHERE TO COPPER SEAL, BECAME LOOSENED BY SOIL AND WAS EASILY REMOVED IN LONG STRIP (AUGUST 12, 1941)

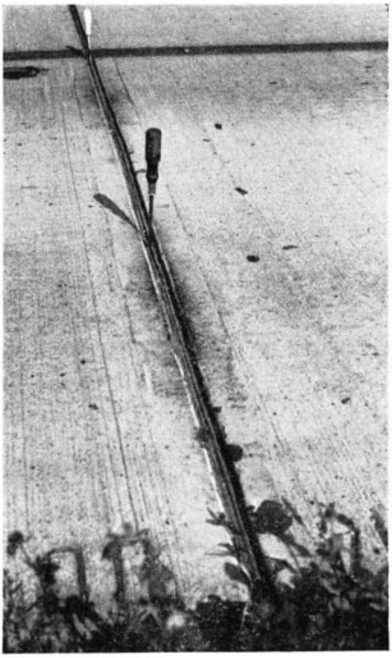
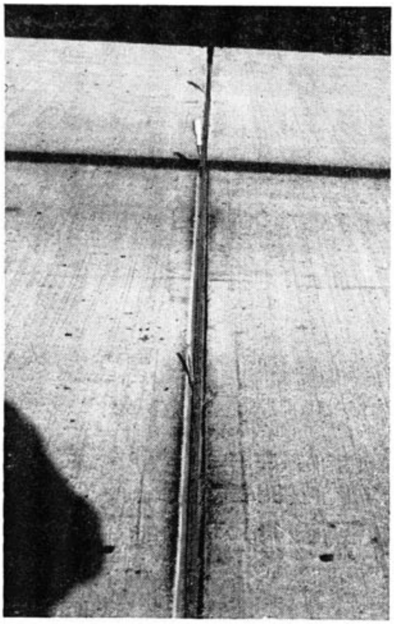


FIG. 113. J-5 EXPANSION JOINT AT STA. 14 + 00 WITH ASPHALT CAP REMOVED, SHOWING LONG SPLIT IN COPPER SEAL (AUGUST 25, 1941)



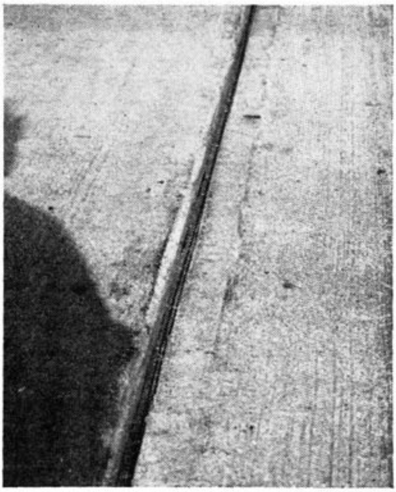


Fig. 115. J-3 Expansion Joint at Sta. 25 + 50 with Asphalt Cap Removed, Showing Splits in Copper Seal (August 25, 1941)

Fig. 114. J-7 Expansion Joint at Sta. 23 + 00 with Asphalt Cap Removed, Showing Long Split in Copper Seal (August 25, 1941)

conclusion appears reasonable from a consideration of the limited movement to which contraction joint seals are subjected and the protection against the action of traffic afforded them by the narrow joint opening.

The copper seals on the expansion joints in the south portion of the experimental road were given a detailed examination on August 22, 1941. These are fiber joints of various makes provided with copper seals over the top. No splits or cracks indicating definite failure were found, but many of the seals were badly crimped, which predisposes the copper to early failure. The observed condition of the seals on individual joints is given in Table 58.

All of the joints of this type consisted of a ½-in. fiber filler. Since they were placed at 25-ft. intervals with no intervening contraction joints, the amount of movement at each joint is less than that at the wider expansion joints at longer spacings. This fact, and the effect of the fiber filler in reducing the number of sharp bends and crimps in the copper, were apparently the reasons for the better performance of the seals on these joints.

TABLE 58

RESULTS OF COPPER SEAL SURVEY

(Armington Experimental Road)

Expansion Joints—South Section—Sta. 68+25 to Sta. 79+50—August 22, 1941

Station	Type	Condition
68+25	J-10	Tending to crimp on long side to inverted V or to M shape.
+50		Same as above. Dented by traffic.
+75		Distinct V crimp. Traffic dents.
69+00		Same as above.
+25		Some V and M crimping. Short side loose in places.
+50		Same as above. Traffic dents.
+75		Same as above.
70+00		Same as above. Short side out of concrete for 12 in. east of C.L.
+25		V crimp forming.
+50		Same as above.
70+75	J-8	(Misstamped 75+75 in pavement) Apparently OK.
71+00		V crimp ⅛ in. high.
+25		V crimp ¼ in. high, 90° angle at top. Closure about 0.2 in.
+50		M crimp for 2 ft. ± middle east lane. Remainder V crimp.
+75		V crimp forming. Old test puncture 2 ft. from west edge. Fiber filler disintegrating. Water in joint.
72+00		V crimp forming.
+25		V crimp except 1½ ft. at east end.
+50		Small V crimp. Badly dented about 3½ ft. east of C.L.
+75		Small V crimp.
73+00		Small V crimp. Punctured seal middle west lane. Fiber filler fair condition. Water in lower part of joint.
77+25	J-9	Small crimp. Loose and dented for about 2 ft. beginning about 1½ ft. from east end.
+50		M crimp about ½ in. wide.
+75		Slight crimp.
78+00		Approximately normal and O.K.
+25		Same as above.
+50		Slight crimp. Punctured seal 1 ft. from east edge. Fiber filler O.K. No free water.
+75		Slight crimp.
79+00		Similar to 77+50.
+25		Slight M crimp.
+50		Moderate M crimp.

Subsequent examinations made at later dates, to determine the condition of the copper seals on all the joints on the experimental sections, were of a qualitative rather than a detailed nature. They were made for the purpose of observing in a general way whether the condition of the seals had become progressively worse. Such observations were made in October, 1941, February, 1943, July, 1943, November, 1943, and August, 1944. These surveys indicated progressive increases in the failures in practically all of the seals.

The condition of the seals on the metal air-chamber joints, as observed July 23, 1943, is given in Table 59. In most cases failures had extended and some of the seals had failed throughout their entire length. Only five of the 37 joints showed no observable failures, and it was believed that failures probably existed in three of these cases, although the presence of asphalt in the tightly crimpled folds of the copper prevented close examination.

TABLE 59

RESULTS OF COPPER SEAL SURVEY

(Armington Experimental Road)

Expansion Joints—North Section—Sta. 2+00 to Sta. 26+00—July 23, 1943

Station	Type	Condition
2+75	J-1	Crimped tight. Failed both tracks east lane.
3+50		Crimped tight. Extend of failure uncertain.
4+25		Crimped tight. Remnants of asphalt trapped; hence impossible to determine failures. Probably failed.
5+00		Previous failures extended, failures in east lane.
5+75		Practically complete failure. Pronounced "pumping" at west end.
6+50	J-4	Complete failure. Weeds growing in joint.
7+25		Previous failures extended.
8+00		Inspection impossible as before. Probably failed.
8+75		Previous failures extended.
9+50		Previous failures extended.
10+25	J-6	Previous failures extended.
11+00		Previous failures extended.
11+75		Previous failures extended.
12+00		No failures observable.
12+25		No failures observable.
12+50		No failures observable.
12+75		Failure in west track west lane.
13+00		No failure observable.
13+25		Complete failure west lane, half of east lane.
14+00	J-5	Practically complete failure.
14+75		Previous failures extended.
15+50		Additional failures, others extended.
16+25		Previous failures extended.
17+00		Practically complete failure.
17+75	J-2	Previous failure extended, additional failures.
18+50		Several short failures.
19+25		Previous failures extended, additional failures.
20+00		Previous failures extended, additional failures.
20+75		Crimp and asphalt prevented accurate inspection. Probably failed.
21+50	J-7	Practically complete failure.
22+25		Failures considerably extended.
23+00		Practically complete failure.
23+75		Practically complete failure.
24+50		Practically complete failure.
25+00	J-3	Previous failures extended.
25+50		Previous failures extended.
26+00		Previous failures extended.

The results of an examination made July 23, 1943, of the copper-sealed fiber joints in the south portion of the road, are given in Table 60. A rapid examination of these joints in October, 1941, had shown that at least one seal on each type of joint had begun to fail. The remarks given in Table 60 are in reference to those in Table 58. For example, a notation in Table 60 of "No apparent change" indicates the seal was in the same condition as recorded in Table 58. It will be noted from Table 60 that the seals on eight of the 30 joints had developed splits in the copper.

While no splits had occurred in the J-10 seals, it is particularly significant that the flange on the short side of the seal had become loose in the concrete rather generally, and, in some cases, had pulled com-

TABLE 60

RESULTS OF COPPER SEAL SURVEY

(Armington Experimental Road)

Expansion Joints—South Section—Sta. 68+25 to Sta. 100+85—July 23, 1943

Station	Type	Condition
68+25	J-10	Flange on short side of seal loose in places.
68+50		Flange on short side of seal loose in places. Concrete broken out along north side of joint from plate dowel upward for distance of 3 ft. from west end.
68+75		Seal crimped and damaged. Flange on short side of seal loose in places. Concrete broken out along south side of joint from plate dowel upward for distance of 3 ft. from east end.
69+00		No apparent change.
69+25		Flange on short side of seal loose in places. Concrete broken out from plate dowel upward for a distance of 4 ft. from west end.
69+50		Concrete broken out along south side of joint from plate dowel upward for distance of 3 ft. from east end.
69+75		Asphalt cap coming off. Flange on short side of seal loose.
70+00		Flange on short side of seal pulled out for 18 in. east of center line.
70+25		Half of asphalt cap gone. Seal loose in places.
70+50		Half of asphalt cap gone. Seal loose in places.
70+75	J-8	No apparent change.
71+00		No apparent change.
71+25		Six-in. failure near west end.
71+50		Failures in all wheel tracks.
71+75		No apparent change.
72+00		Six-in. failure east wheel path east lane.
72+25		Six-in. failure west wheel path west lane.
72+50		No seal change. Bad spall north side of joint in east lane.
72+75		Failures in all wheel paths.
73+00		Failures in west wheel path of west lane, east wheel path of east lane.
77+25	J-9	No apparent change.
77+50		Six-in. failure west wheel path of west lane.
77+75		Little apparent change, crimped about ⅛ in.
78+00		Little apparent change, crimped about ⅛ in.
78+25		Little apparent change, crimped about ⅛ in.
78+50		Little apparent change, crimped about ⅛ in.
78+75		Little apparent change, crimped about ⅛ in.
79+00		Little apparent change, crimped about ⅛ in.
79+25		Little apparent change, crimped about ⅛ in.
79+50		Seal split about 12 in. at west end where it enters concrete.
100+02.5	J-5	Practically complete failure.
100+85		Practically complete failure.

pletely out of the concrete. Movements of the joint were taken up by sliding of the loose flange, resulting in less flexing of the copper, which probably explains why the copper did not fail. The loosening of the seal in the concrete can reasonably be considered as serious a failure as a split in the copper; in either case the seal is no longer watertight.

It is apparent from these data that copper seals of the types installed in this project, both those on the metal air-chamber joints and those on the metal-sealed fiber joints, develop serious failures in 3 to 5 years, and thus do not possess sufficient service life to justify their use. In considering this conclusion, it should be borne in mind that these results occurred with an almost complete lack of heavy traffic, which experience has shown may be an important factor in the destruction of seals.

FIG. 116

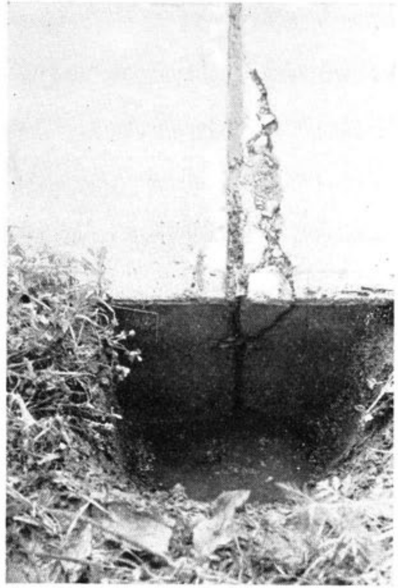


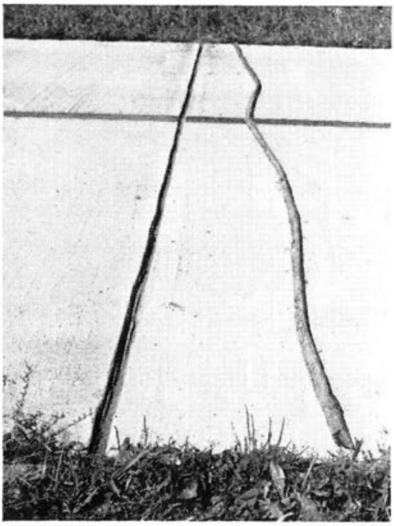
FIG. 117

FIG. 118

FIG. 116. DUMMY JOINT WITH PREMOLDED RUBBER SEAL AT STA. 26 + 25, SHOWING CRACK FORMED AS INTENDED BUT OPENED SO WIDE THAT RUBBER SEAL DROPPED TO BOTTOM OF GROOVE (JULY 23, 1941)

FIG. 117. J-10 EXPANSION JOINT AT STA. 69 + 25, SHOWING FAILURE IN CONCRETE SLAB INDUCED BY PLATE DOWEL (JULY 23, 1941)

FIG. 118. SAME JOINT AS IN FIG. 117, WITH SHATTERED CONCRETE REMOVED (JULY 23, 1941). THIS FAILURE IS TYPICAL OF THOSE PRODUCED IN LABORATORY TESTS OF THIS TYPE OF JOINT



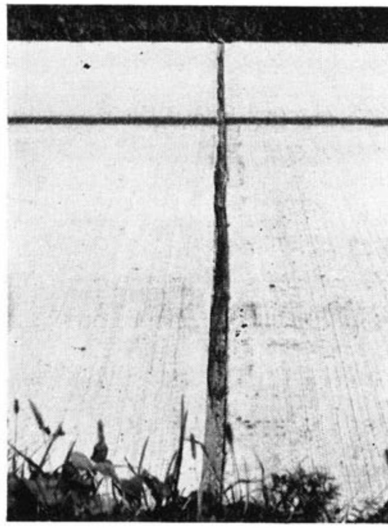
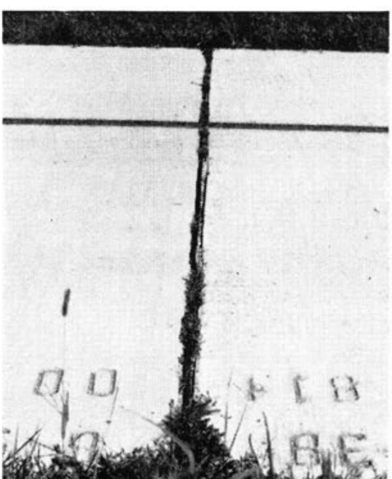


Fig. 119. J-8 Expansion Joint at Sta. 73 + 00, Showing Loose Asphalt Cap Removed to Expose Sharply Crimped Copper Seal (August 12, 1941)

Fig. 120. J-11 Expansion Joint at Sta. 76 + 25. Metal Side Plates Did Not Remain Tight against Filler, Permitting Dirt to Enter between Them and Accumulate in Extrusion Chambers (August 25, 1941)



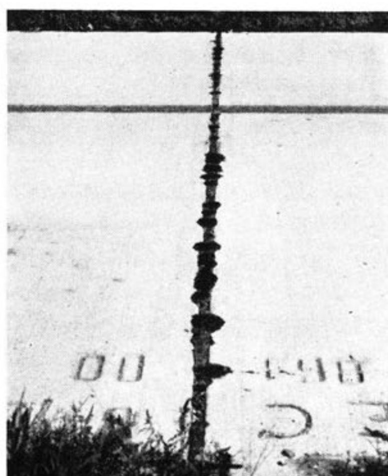


Fig. 121. Preformed Rubber Expansion Joint at Sta. 81 + 00, Showing Weeds Growing in Soil Collected between Loose Filler and Concrete. Rubber Shows Some Deterioration (August 25, 1941)

Fig. 122. Bituminous Premolded Expansion Joint at Sta. 86 + 00, Showing Slight Extrusion. Filler in Good Condition (August 25, 1941)

(f) Asphalt Seals

Two types of asphalt seals were used on the expansion joints in the experimental road; a premolded mastic cap was furnished on some of the metal air-chamber joints, and the conventional poured asphalt seal was used on the remainder of the metal air-chamber joints and the joints with fiber fillers.

It developed that the only practical value of the premolded cap was that it served to protect the seal during the paving operations and provided a guide for edging the joint. After only a few months, many of these caps began to show signs of deterioration and no longer adhered to the seal, the first failure occurring generally near the edges and centerline of the pavement. By August, 1941, when all the caps were removed to permit inspection of the copper seals, most of the premolded ones were loose and some had been displaced by the action of traffic. Those which were still in place were broken and split and could be easily lifted out of the joint opening. The asphalt had not adhered to the copper or the concrete, and soil had worked under the caps, causing them to be heaved up to a point where they were subjected to severe punishment from wheels of vehicles, which hastened their deterioration. The cap on the J-1 joints, which fit the inverted U-shaped seal like a saddle, split lengthwise along the top, due to the bending occurring when the joint closed.

The asphalt seals, formed by filling the space above the copper seal on the air-chamber joints with hot asphalt, were also found to be inadequate. Like the premolded type, they did not adhere to the copper and thus permitted soil and water to infiltrate around them. They were more plastic than the premolded type and hence, at least until they absorbed a considerable amount of foreign matter, conformed themselves more readily to the changes in the shape of the seal brought about by movement of the joint. There was some evidence indicating that the poured asphalt which was held in the trough of the M-shaped seal after the seal was pinched by closure of the joint, formed a rather effective temporary seal.

Poured asphalt seals on expansion joints with fiber fillers were in reasonably good condition at the end of three years. They appeared fairly effective in keeping out soil but they did not prevent infiltration of water.

Poured seals on dummy joints were in good condition at the end of three years' service. In most cases there had been a little extrusion and the kneading of this excess material by traffic resulted in a rather effective seal.

The premolded asphalt strips which had been inserted in some of the dummy joints at the time of construction were in good condition after three years' service. The effectiveness of these had also been improved by the kneading action of traffic. The principal advantage of this type of construction appears to be that it insures the groove being full depth.

(g) Premolded Rubber Seals

The premolded type of rubber seal is definitely unsatisfactory. At the end of three years the seal from one of the expansion joints had disappeared. According to the maintenance patrolman, water freezing in the joint space during the winter of 1939-40 had forced the seal part way out. It is reported to have disappeared from the joint during the winter of 1940-41, and, although this could not be verified, it is believed that the seal was torn out by a snow plow. The top seals on the other four joints appeared to be in good condition, but the end seals were loose and had permitted soil and water to enter the joint space.

Three of the five rubber contraction joint seals were in fairly good condition except for a little hardening and deterioration of the rubber. The other two were loose, one having dropped down to the bottom of the joint groove.

Subsequent examinations of the rubber seals made in 1943 and 1944 showed that the rubber was rapidly losing its elasticity and had become permanently compressed by closure of the joint during periods of sustained expansion of the concrete. On February 4, 1943, all of the seals in expansion and contraction joints were loose and completely ineffective. On July 23, 1943, two of the seals from expansion joints were completely out of the joint opening. The remaining two were tight due to closure of the joints during the hot weather. The seals on all the contraction joints were loose and two had dropped to the bottom of the groove. In August, 1944, the rubber seals were either entirely gone, badly compressed, or loose, and showed signs of further progressive deterioration.

(h) Dummy Joint Breaks

Dummy or weakened plane joints were used as contraction joints between some of the expansion joints in the north section of the experimental road. A 1,120-ft. length of pavement in the south section, separated from the adjacent portions by 4-in. open joints, was provided with contiguous dummy joints at spacings of 20, 25, 30, and 35 ft., in order to study their effectiveness in preventing irregular transverse cracks. As explained in Section 17 (page 201), these joints were formed by cutting a groove with a T-iron. The depth of the groove was varied, 1-, 1½-, 2-, and 2½-in. depths being used, in order to determine the minimum depth which will insure that the slab will crack in the groove.

Part of these dummy joints had a premolded ribbon-type filler inserted as soon as the groove was cut in the fresh concrete, insuring a definite depth. The remainder were formed by cutting a groove in the fresh concrete with a T-iron and inserting a piece of weatherboarding to preserve the groove until the concrete had set and to serve as a guide for edging. Measurements of the depth of the grooves of the latter joints were taken before the asphalt filler was applied. The average results are given in Table 61. It will be noted that the average depth of the 1-, 1½-, and 2-in. grooves was about ½ in. less than the nominal depth. The average depth of the 2½-in. grooves was about ¼ in. greater than the nominal depth, for no apparent reason.

In October, 1938, a rapid survey showed that the pavement had cracked in every dummy joint, irrespective of the depth of the groove. A detailed survey was made July 23, 1941, to determine the regularity, nature, and angularity of cracks in the slabs. A summary of the results is given in Table 61.

The deviation of the cracks from the vertical varied inversely with the depth of the groove, the greatest angularity occurring with the 1- and 1½-in. grooves and the least with the 2½-in. groove. It is doubtful that these findings are of any particular significance; certainly nothing has developed at these joints to indicate that angularity of the break, or lack of it, is a benefit. It was observed that the

TABLE 61
SUMMARY OF CRACKS FORMED BY DUMMY JOINTS
(Armington Experimental Road)

Nominal Depth	Number of Ends	Angle with Horizontal						Actual Depth (Joints with poured fillers only)			Net Area	Eccentricity
		90°		80°-90°		70°-80°						
in.		Number	Percentage	Number	Percentage	Number	Percentage	Maximum	Minimum	Average	percentage	in.
1	40	24	60.0	13	32.5	3	7.5	1⅜	⅝	1 1/32	86.0	½
1½	30	18	60.0	8	26.7	4	13.3	1¾	⅞	1⅛	78.5	¾
2	60	47	78.3	13	21.7	..	...	2½	1½	2 1/32	71.5	1
2½	30	26	86.7	4	13.3	..	...	2⅞	2½	2 21/32	64.3	1¼
All	160	115	71.8	38	23.8	7	4.4	...	...	....	....	..

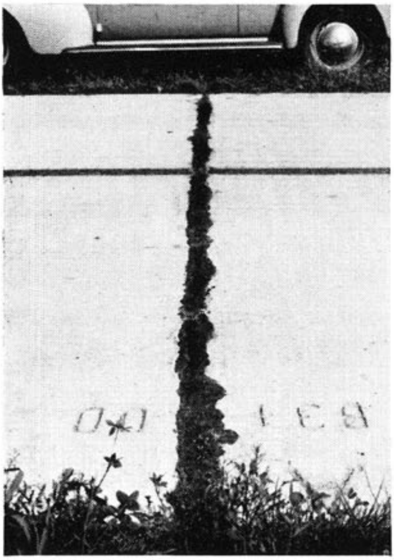
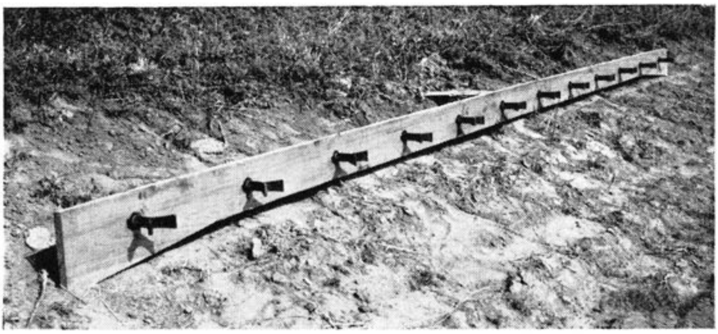


FIG. 123 (AT LEFT). BITUMINOUS PREMOLDED EXPANSION JOINT AT STA. 83 + 00, SHOWING EXCESSIVE EXTRUSION CAUSED BY POURING ASPHALT SEAL ON TOP OF JOINT. COMPARE WITH JOINT WITHOUT SEAL IN FIG. 122 (AUGUST 25, 1941)

FIG. 124 (BELOW). WOOD EXPANSION JOINT, CONSISTING OF 1-IN. CLEAR CYPRESS BOARD WITH L-1 LOAD TRANSMISSION DEVICE, ASSEMBLED READY FOR INSTALLATION (JUNE 16, 1938)



direction of the break at one end was no indication of the direction at the other end. Probably the break, in taking an indirect course across the pavement, changed its vertical direction numerous times.

Part of the survey was made immediately following a heavy shower. Water was observed running freely from many of the joints for several minutes, and seepage continued for more than an hour. At several joints, water flowed from under the slab for a distance of several feet upgrade from the joint.

Table 61 also shows the reduction in cross section and the eccentricity effected by grooves of various depths, based on the nominal depth of the groove. The eccentricity is computed by taking the difference between mid-depths of the full cross section and the reduced cross section. It is apparent that the dummy joint results in an appreciable reduction in the cross section which must resist expansive forces in a pavement, and introduces considerable eccentricity in the line of action of the resultant of these forces with respect to the center of the full cross section. These may be factors in the ultimate behavior of the slab, but so far there are no indications that dummy joints have introduced any objectionable weakness in the pavement.

(i) Natural Transverse Cracks

It had been determined definitely from experience that joints installed at 30-ft. intervals, such as was done in Illinois from 1933 to 1937, inclusive, would not prevent or even retard the formation of transverse cracks. Consequently, in building the experimental sections variable joint spacings were used for the purpose of studying the effect of joint spacing on cracking. Spacings of 15, 20, 25, 27.5, 30, 35, and 50 ft. were used. A majority of the panels were made 25 ft., in an effort to determine whether this length is more effective than the 30-ft. length. Six of the 50-ft. panels were reinforced with wire mesh of various weights; the other five contained no reinforcement.

Although no detailed crack survey was made until July 8, 1941, the experimental sections were examined for transverse cracks each time the road was surveyed. The first cracks were observed on September 28, 1939, one at Sta. 74 + 85 in a 25-ft. panel between two nonextrusion type joints, and one at Sta. 87 + 35 between two ½-in. bituminous premolded joints in a 50-ft. nonreinforced panel. Two additional cracks were observed in 25-ft. panels on October 17, 1939, one at Sta. 7 + 85 between J-4 expansion and contraction joints and the other at Sta. 22 + 37 between J-7 expansion and contraction joints. Three more cracks were discovered on January 9, 1940. One was located at Sta. 7 + 68 in a 25-ft. panel between two J-4 contraction joints. The other two were in the east lane only at Sta. 87 + 05 and Sta. 87 + 15, in the same 50-ft. unreinforced panel in which a crack was recorded on September 28, 1939.

A detailed survey made on July 8, 1941, included an examination of both experimental sections and the intervening section containing J-5 joints placed at 30-ft. intervals under the regular contract. No new cracks were found in the north experimental section, but four had developed in the south experimental section at these approximate locations: Sta. 70 + 35 in a 25-ft. panel between a J-8 and a J-10 joint; Sta. 78 + 12 in a 25-ft. panel between two J-9 joints; Sta. 103 + 10 and Sta. 104 + 85, both in 30-ft. panels between two metal contraction joints in the wood joint subsection.

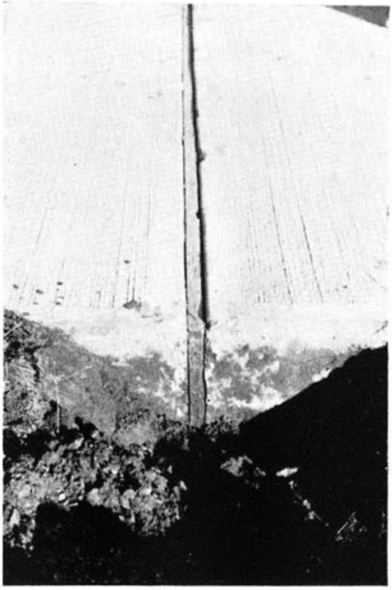


FIG. 125. WOOD EXPANSION JOINT AT STA. 102 + 65. BOARD IS SOUND AND TIGHT IN JOINT WITH VERY THIN LAYER OF SILT ON EACH SIDE. TOP WITHOUT ASPHALT SEAL IS ONLY SLIGHTLY SCARRED BY TRAFFIC ABRASION (OCTOBER 10, 1939)

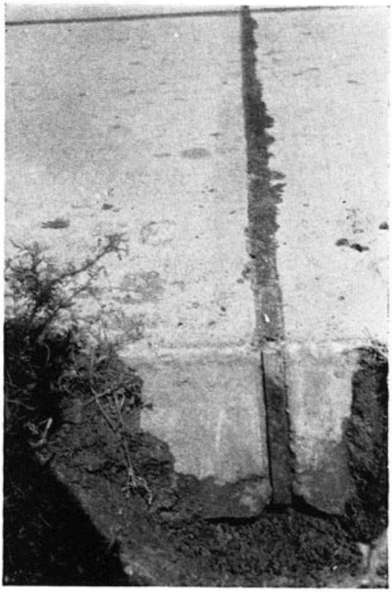


FIG. 126. WOOD EXPANSION JOINT AT STA. 105 + 35, SHOWING ASPHALT SEAL, ADDED SHORTLY AFTER INSTALLATION, STILL IN GOOD CONDITION. WOOD SOUND AND PRACTICALLY FULL THICKNESS. TIGHT ON RIGHT SIDE, 1/32-IN. LAYER OF SILT ON LEFT SIDE (JULY 23, 1941)

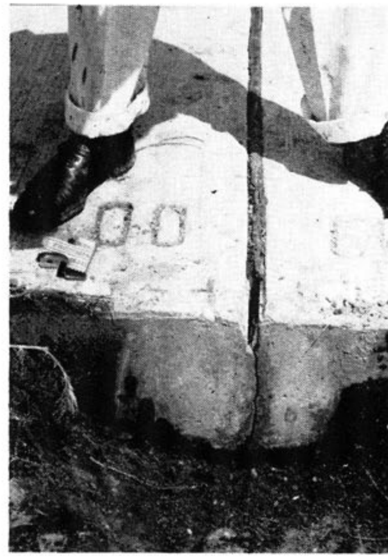


Fig. 127. Dummy Contraction Joint at Sta. 33 + 00, Showing Typical Crack Formed by 2½-in. Groove (July 23, 1941)

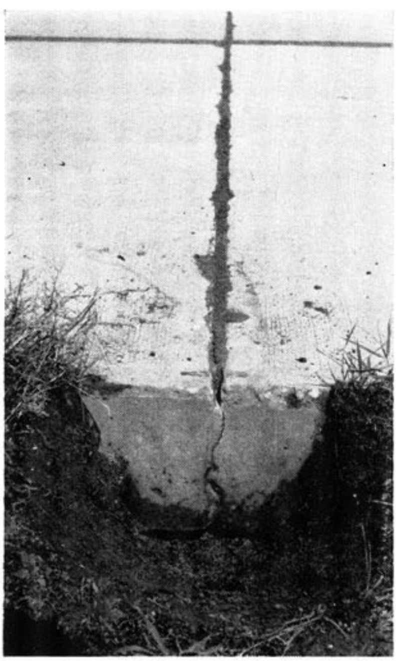
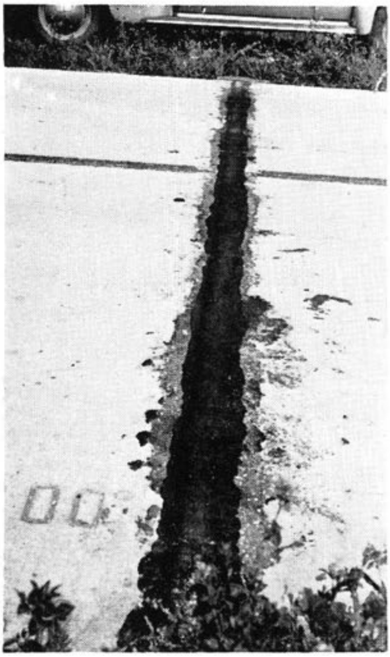


Fig. 128. Dummy Contraction Joint at Sta. 98 + 40, Showing Typical Crack Formed by 1-in. Groove (July 23, 1941)



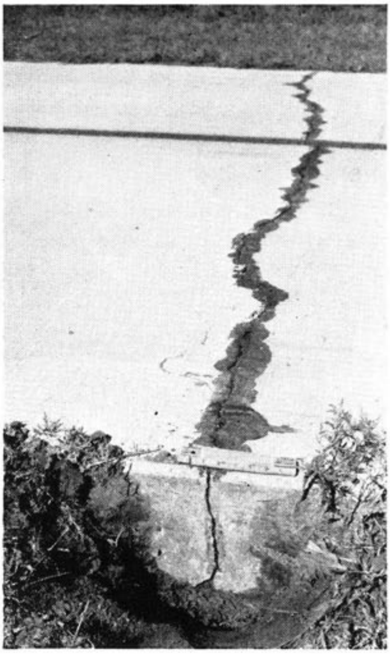


FIG. 129. VIEW SHOWING CONDITION OF 4-IN. OPEN JOINT AT STA. 88 + 00 ON JULY 8, 1941. APPROXIMATE CLOSURE SINCE INSTALLATION ON JUNE 13, 1938, 2.8 IN.

FIG. 130. TRANSVERSE CRACK AT STA. 74 + 85. CRACK OPENING IS ABOUT ¼ IN. WHAT APPEARS TO BE EXTRUSION ACTUALLY IS AN EXCESS OF ASPHALT POURED ON CONCRETE IN ATTEMPTING TO FILL THE CRACK (AUGUST 12, 1941)

An additional crack was found on July 15, 1941, at Sta. 25 + 35 in a 25-ft. panel between J-3 expansion and contraction joints. Two more cracks had developed by August 12, 1941, one at Sta. 25 + 12 in a 25-ft. panel formed by J-3 expansion and contraction joints, and the other in the west lane only at Sta. 38 + 65 in a 15-ft. panel between a dummy contraction joint and a fiber expansion joint. The crack which had previously been noted in the east lane at Sta. 87 + 15 had also progressed across the west lane. On July 23, 1943, one additional crack was observed in the west lane only at Sta. 72 + 87 in a 25-ft. panel between two J-8 joints. This crack had progressed across the east lane on November 18, 1943, and a new crack was found at Sta. 8 + 62 in a panel between J-4 expansion and contraction joints. The most recent examination was made August 16, 1944, at which time one new crack was found at Sta. 79 + 37 in a 25-ft. panel in the J-9 joint subsection.

Table 62 summarizes the cracking occurring in the two experimental sections through August 12, 1941, and in the regular contract section through July 8, 1941. While the lengths of the various subsections are in most cases not sufficient, nor the pavement old enough to justify definite conclusions, general observations are of interest. There is an indication that the amount of cracking was proportional to the length of panel. None of the 20-ft. panels had developed cracks at three years, the 25-ft. panels were cracked less than 30-ft. panels, and the 30-ft. panels had proportionately fewer cracks than the 50-ft. nonreinforced panels, although there were not enough of the latter to be definitely significant. Cracking in the 15-ft. panels was disproportionately high, one crack having developed in ten panels, but it is believed that this crack, because of its location and diagonal direction, was caused by some unusual condition which was not apparent. While cracking probably is proportional to length of panel, it is nevertheless significant that the combined number of joints and cracks is inversely proportional to the length of panel. In other words, natural cracks are prevented from occurring by what amounts to building controlled cracks into pavements. As pointed out elsewhere in this bulletin, this may be a questionable procedure because it may reduce the effective service life of a pavement.

Considerably more cracking occurred in the 30-ft. panels on the regular contract section than in panels of equal length on the two experimental sections. This finding probably is of no particular significance since the topography of the contract section, being more rolling and requiring more cut and fill, is probably more conducive to transverse cracking than that of the experimental sections, which were purposely located to avoid grades, cuts, and fills as much as possible.

(j) Extrusion of Fillers and Bituminous Seals

Extrusion of joint fillers and bituminous caps and seals is objectionable because it contributes to the poor riding qualities of a pavement. This subject is discussed in some detail elsewhere in this bulletin (page 171), and measurements obtained from field surveys on regular pavements are analyzed. No measurements of extrusion were made at the joints in the Armington Experimental Road, but certain observations may be made on the basis of visual examinations.

The asphalt seals on the fiber joints showed only slight extrusion, and the small amount of sealing material rising above the surface of the pavement was pounded down by traffic, apparently improving the sealing effect.

Considerable extrusion occurred at all-metal expansion joints, the

TABLE 62

SUMMARY OF RESULTS OF NATURAL TRANSVERSE CRACK SURVEY

(Armington Experimental Road)

(Includes cracks in experimental sections through August 12, 1941; in contract section through July 8, 1941)

Station		Number of Panels	Length of Panels	Total Length	Number of Cracked Panels	Percentage of Cracked Panels	Number of Cracks	Computed per Mile			Notes
From	To							Joints	Cracks	Joints and cracks	
FIRST EXPERIMENTAL SECTION											
2+00	37+25	141	25	3,525	5	3.5	5	211	8	219	
37+25	38+75	10	15	150	1	10.0	1	352	35	387	
SECOND EXPERIMENTAL SECTION											
68+00	79+50	46	25[1]	1,150	3	6.5	3	211	14	225	
79+50	82+50	10	30[2]	300	0	0	0	176	0	176	
32+50	85+50	6[3]	50	300	0	0	0	106	0	106	Mesh reinforced
85+50	88+00	5[3]	50	250	1	20.0	3	106	63	169	Not reinforced
88+00	89+75	5[3]	35	175	0	0	0	151	0	151	
89+75	91+25	5[3]	30[2]	150	0	0	0	176	0	176	
91+25	95+00	15	25[1]	375	0	0	0	211	0	211	
95+00	99+20	21	20	420	0	0	0	264	0	264	
99+20	100+85	6[3]	27.5	165	0	0	0	192	0	192	
100+85	105+35	15	30[2]	450	2	13.3	2	176	23	199	
[1]Combined		61	25	1,525	3	4.9	3	211	10	221	
[2]Combined		30	30	900	2	6.7	2	176	12	188	
CONTRACT SECTION											
38+75	68+00	89	30	2,670[4]	43	48.3	43	176	85	261	

[1] See "[1]Combined" in table.
[2] See "[2]Combined" in table.
[3] Number of panels too small to be definitely significant.
[4] Omits bridge.

bituminous cap being forced up by the closure of the joint and by soil working down between the copper seal and the cap.

The extrusion of the 1-in. bituminous premolded joints was excessive, but its effect was modified considerably by the action of traffic, and the roughness was not noticeable at ordinary speeds and hence not very objectionable. The ½-in. bituminous premolded joints showed practically no extrusion except where an asphalt seal had been applied to the top of the joint.

Paradoxically, the greatest extrusion in preformed joints occurred at the so-called non-extruding joints. It was observed that the extrusion chambers did not perform effectively, very little of the premolded filler entering the chambers provided for that purpose. Soil entering the joint between the side walls and the filler, and into the expansion chambers, added to the poor performance. In some places the filler had been forced out of the joint to a height of 1¼ in.

The greatest extrusion occurred at the 4-in. poured joints. This was to be expected because of the large volume of material in these joints, and the large movements which occur. It was reported that maintenance patrolmen had trimmed off the extruded material several times prior to July 8, 1941. The filler became very soft at summer temperatures, flowed out of the ends of the joints onto the shoulders, and spread out on the surface of the pavement along the joints. Stains on the pavement indicated that the extruded material had spread out 6 to 8 in. on each side of the joints.

The effect of temperature in causing extrusion at 4-in. joints was illustrated during a survey made August 22, 1941. At 7:00 a.m., when the air temperature was 66 deg. F., it was noted that the filler at the middle of the traffic lanes was depressed about ½ in. below the surface of the pavement and at the end of the joints was about ¼ in. above the slab. At 12:30 p.m., when the air temperature had risen to 85 deg. F., the filler was about ⅜ in. high at the ends of the joints and nearly level with the pavement surface at the middle of the traffic lanes. During that time the average closure of the joints was about $\frac{1}{16}$ in. It is apparent that small closures of the joint and expansion of the filler material will result in an undesirable amount of extrusion in wide joints filled with the type of asphalt used on this project.

Much excess filler was observed at transverse cracks, but this seemed to have resulted from applying too much material at the time the cracks were poured rather than from extrusion. It appears that improved technique in filling cracks would conserve material, provide a smoother surface at cracks, and improve the appearance of the roadway.

(k) Daily Movements at Joints

Daily movements may be expected at joints as the concrete expands and contracts with changes in its temperature. With closely-spaced expansion joints such movements are relatively small and of little importance when the joints are of the premolded type. However, such daily movements, although small, may influence the failure of copper seals, due to the continual flexing of the copper.

While no large-scale study of these movements was made during this investigation, a few measurements made at 4-in. open joints indicate the magnitude of daily cycles. On August 13, 1941, the two 4-in. joints each closed almost exactly 0.10 in. between 7:30 a.m. and 12:30 p.m. Temperatures of the air, pavement surface, and subgrade at 7:30 a.m. were 60, 66, and 78 deg. F., and at 12:30 p.m. 76, 81 and 80 deg. F. A rise in these temperatures to 77, 84, and 83 deg. F., respectively, at 3:30 p.m. was accompanied by a further closure of 0.01 in. On August 22, 1941, the joints closed an average of 0.04 in. between 7:00 a.m. and 10:30 a.m., with a rise in air temperature from 66 to 79 deg. F. At 12:30 p.m., with the air temperature at 85 deg. F., the average closure had increased to 0.07 in.

(l) Infiltration

It has been emphasized throughout this bulletin that infiltration of foreign material into joints and transverse cracks is one of the principal reasons why expansion joints are required and is a large factor in the progressive growth of pavements and the resulting closing of expansion joints. This experimental pavement is not of sufficient age for excessive infiltration to have occurred, but the general indications observed from an examination made in August, 1941, are that all of the joints are more or less subject to this undesirable action.

It was evident that water at some time had filled the metal joints, and dirt was found to have collected in the bottom of the metal stools. Soil was found to have collected in a limited amount along the premolded fiber joints and to an even lesser degree along the bituminous premolded joints. Four of the wood joints were tight and free from infiltration, and one had about $\frac{1}{32}$ in. of soil between one side of the board and the concrete. On the whole, infiltration in all these joints had not developed to serious proportions. About $\frac{1}{2}$ in. of soil had collected in the joint that had lost its upper premolded rubber seal. Soil had collected in several of the transverse cracks and dummy joints. The greatest accumulation of soil was found along the sides and in the extrusion chambers of the so-called non-extruding joint.

(m) Watertightness

None of the joints can be said to be watertight even at early ages. The metal-sealed joints were relatively tight at first, but when the copper seals failed and the sheet steel bodies rusted out, these joints became reservoirs for collecting water and channels for the distribution of water to the subgrade. In this respect they were the poorest type on the project.

The fiber joints, being porous, collected water even when provided with a bituminous seal and conveyed it to the subgrade. However, the rate of transmission was much slower than in the case of metal joints. Both the wood and the bituminous premolded joints appeared to be more effective than other types in keeping water from infiltrating to the subgrade.

A rough test was made to study the relative watertightness of various joints by pouring water in the joint groove and observing how long the water remained. The water ran into the metal joints on which the seals had failed almost as rapidly as it could be poured. The fiber joints took water very rapidly at first but the rate of absorption slowed down in a short time. The bituminous premolded fillers and wood joints had the lowest rate of absorption.

The asphalt seal applied to cracks did not appear to be entirely effective in preventing the infiltration of water. Only a little water appeared to pass the seals on the top of the dummy joints, but water entered these joints from the ends and the bottom. In general, it appeared that the use of asphalt on joints was of some value in reducing the percolation of water, because of its sealing effect and the fact that it limits the amount of water which can collect in the top of the joint.

(n) Reinforcement

Six 50-ft. panels were reinforced with wire mesh reinforcement. Three panels contained mesh with No. 4 wires spaced 5 in. by 9 in., weighing 54 lb. per 100 sq. ft. Two panels were reinforced with mesh with No. 3 wires spaced 6 in. by 12 in., weighing 51 lb. per 100 sq. ft. The other panel contained mesh with No. 3 wires spaced 6 in. by 6 in., weighing 68 lb. per 100 sq. ft. No cracks had occurred in any of these reinforced panels up to August 16, 1944. However, judging from the cracking which occurred in the somewhat similarly reinforced pavements built between 1938 and 1940, it is reasonable to expect that cracks will eventually occur in these experimental panels. On the basis of the same experience, it is also reasonable to expect that such cracks as do form will be held tightly closed as long as the wire mesh remains intact.

(o) Load Transmission Devices

Load transmission devices were considered primarily from the standpoint of installation, no method of measuring or gaging their effectiveness in transferring load and distributing stress being available. The two principal problems, encountered to a greater or lesser degree in the installation of all the load transmission devices, were keeping the devices in proper alignment and preventing the formation of honeycombed concrete adjacent to them. As mentioned previously, the wood joints were much more effective than all other types in providing proper alignment.

VI. Cutting Joints with Abrasive Wheels

19. *General.*—A number of objectionable conditions arise from the conventional methods of installing joints in concrete pavements. Among these is the tendency of the surface of the pavement immediately adjacent to joints to be built high, due to the necessity of stopping the finishing machine and raising the screeds at the joint. This is discussed in Section 15(g) (page 170).

Another condition is the inferior concrete deposited immediately adjacent to joints, resulting from the finishing machine dragging excessive laitance up against the joint and from the excessive working of the concrete at this location in edging along the joint. In practice, it is difficult to edge the joints at exactly the right time, and the already inferior concrete is further damaged by excessive troweling. Furthermore, there is a tendency to trowel so that the edges along the joint are higher than the surrounding surface, which makes these edges more susceptible to traffic abrasion. It is only necessary to observe the great amount of spalling and ravelling which occur, particularly at expansion joints, to conclude that the construction procedure employed for the installation of joints is unsatisfactory.

There is also the inefficiency of the usual operation to be considered. Stopping the finishing machine at each joint, shoveling the excess concrete away from in front of the screed, raising the screed, and moving the machine over the joint, all take time and labor and slow up the progress of the finishing operations. The amount of hand finishing that must be done at each joint also adds considerably to the cost of construction, particularly since part of this work must be done after the end of regular working hours and paid for at overtime rates.

It was with this knowledge that Professor W. C. Huntington, Head of the Department of Civil Engineering, University of Illinois, in 1938 conceived the idea of cutting joints in pavements with an abrasive wheel. The suggested method appeared to have so many possible advantages that the University undertook as a research project the development of a machine for doing this work. Earlier work of this nature had been done in France, Germany, and California, but knowledge of these investigations did not come to the attention of the University until after its machine had been developed. The work was conducted with funds furnished by the Illinois Division of Highways.

20. *Development of Machine.*—The first machine developed at the University, designed for use in the laboratory, consisted of a movable

carriage which supported the driving unit and rolled on four wheels along a structural steel frame forming a track. The cutting wheel was mounted on the end of an adjustable arm which could be set for various depths of cut. The wheel was driven by an electric motor through a V-belt drive. In Fig. 131 the machine is shown as set up for cutting a groove in a laboratory specimen.

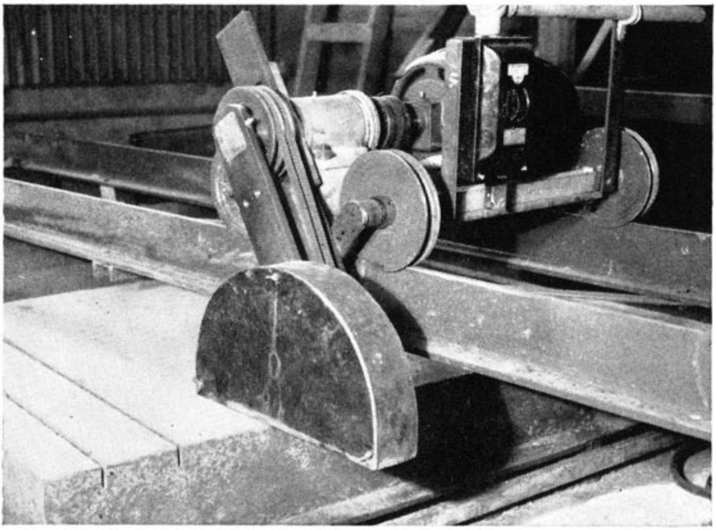


FIG. 131. VIEW OF LABORATORY JOINT CUTTING MACHINE

Another machine was built later for use in cutting joints on actual construction projects. Basically it was of the same design as the original but, because electric power is not always available on construction jobs, the second machine was equipped with a gasoline engine. It was also equipped with two cutting wheels, one set to the rear and to one side of the other in order to cut two grooves simultaneously. The lateral distance between wheels was adjustable to various widths up to 1 in. The wheels could also be adjusted to operate in tandem; that is, one wheel directly behind the other so that both would cut in the same groove. When operated in this manner, the second wheel was set to cut to a greater depth than the leading wheel. Figure 132 shows this machine being operated on a pavement.

21. *Development of Method.*—Initially it was the intention to cut the joint to the full depth of the slab but this procedure was soon

Fig. 132. Machine Developed for Cutting Joints in Pavement Slabs

found to be impracticable. The time consumed was prohibitive, large diameter wheels were required, and excessive wheel breakage occurred.

It was then decided that the desired results could be obtained by installing conventional joints with a height of 2 in. less than the thickness of the slab, and then cutting out the 2-in. prism of concrete directly above the joint material soon after the concrete had set. The various steps in this procedure are shown in Fig. 133. This method permits the slab to be poured monolithically and the operation of the finishing machine to proceed without being interrupted at each joint.

The cutting wheels generally used in this process, known as "solid" type wheels, are made of granular abrasive material cemented together with a suitable bonding agent into a disc ⅛ in. or more thick. Wheels of ⅛- and ¼-in. thickness were tried in the tests. The ⅛-in. size was more satisfactory, but this might not have been true had a more powerful driving motor been used. Many varieties of wheels are made with different grades and types of abrasives and bonding materials. A considerable amount of laboratory work was performed in determining the results to be obtained with different wheels and in finding suitable wheels for the job. During the period of the investigation a number of different wheels were tried with varying success.

At first a whole day was required for cutting one groove to a depth of 2 in. in an 11-ft. slab. Later this time was reduced to 2 hr. and finally to 11 min., or 1 ft. per min. This reduction in time was not

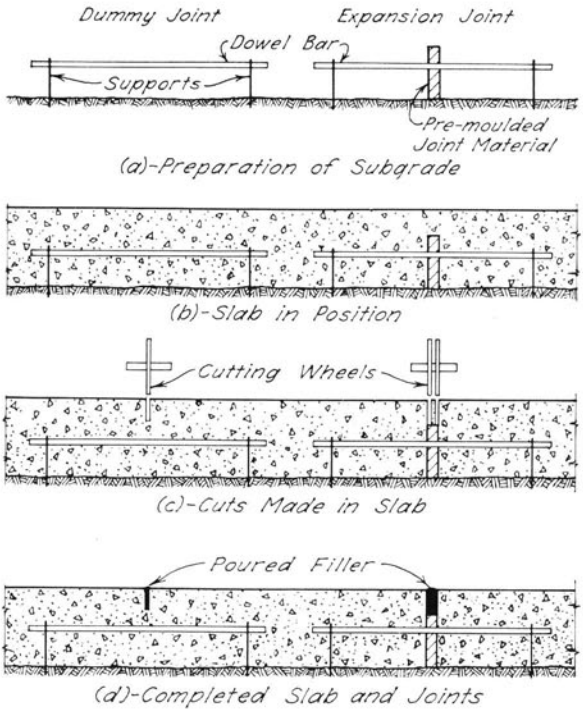


FIG. 133. PROCESSES IN JOINT CONSTRUCTION IN A CONCRETE PAVEMENT, USING A ROTATING CUTTING WHEEL

due entirely to difference in wheels; other improvements in the equipment and changes in technique were largely responsible.

The solid type wheels are very brittle, any lateral pressure causing them to shatter. The machine and the track on which it runs must be constructed so that the carriage will travel in a straight line without any twist or motion sideways. Even with these precautions, extreme care must be taken during the operation of the machine in order to avoid excessive wheel breakage. A steel-center wheel, consisting of a steel disc with a rim of abrasive material cemented to its periphery, was tried out during the tests. It appeared that breakage could be largely eliminated by the use of this type of wheel. However, these wheels cost several times as much as the solid type and are somewhat thicker. More work would have to be done to determine which type is the more economical.

The best results were obtained on concrete containing crushed stone coarse aggregate. It cut relatively fast, either wet or dry, with little wheel wear and a minimum of wheel breakage. On the other hand, gravel coarse aggregate concrete was very difficult to cut. Even when water was used, the work was slow and wheel wear and breakage were excessive. There was some indication during the investigation that a high velocity stream of water, properly directed to wash away the detritus and keep the pores of the wheel from clogging, will result in greater cutting speed and sharply reduce wheel wear and breakage when cutting gravel concrete.

22. *Field Experience.*—By arrangement with the Illinois Division of Highways, joints were cut in three contract sections during 1938 and 1939. Six joints were cut on Route 1 in a two-mile length of pavement starting about one mile north of Danville, Illinois. Thirty joints were cut on F.A. Route 135, Section 10, between Cisco and Monticello. Fifteen joints were cut on F.A. Route 161, Section 10-H, south of Mt. Pulaski.

The first joints were cut in gravel concrete in the pavement north of Danville. As is the case in most development work, many unexpected difficulties were encountered which reduced production and affected results.

The machine suffered an accident while it was being transported to the site of the work which caused it to vibrate while operating. The man in charge of the work reported that in his opinion this vibration was the cause for otherwise unexplainable breakage of wheels on the job.

It was also discovered that the machine was not properly balanced, and, when cutting joints in pavement on a grade, the vibration of the engine caused one end of the track to drift downgrade. This produced a lateral pressure which broke the cutting wheel. Other mechanical difficulties developed which took the machine out of service for several days while repairs were made.

For maximum production and minimum wear on cutting wheels, it is desirable to cut joints early. But experience on this job indicated that when cutting is done too early in gravel concrete, small pebbles pull out of the cement paste, due to the pressure of the cutting wheel, and lodge in the freshly cut groove behind the wheel, making it impossible to return the wheel through the groove for a new cut. Because of this condition, all afternoon was required on one occasion to cut 11 ft. of joint in concrete 24 hr. old. This condition was also present, but to a lesser extent, when the concrete was 48 hr. old.

Cutting in gravel concrete was done both with and without the use of water on the cutting wheels. Both methods produced good cuts, but those without water required a much longer time and resulted in much greater wheel wear. Some warpage of the wheels occurred when water was used and it was thought that this was due to water being applied to one side of the wheel only.

Two joints were also cut in a part of the Danville section made with crushed stone coarse aggregate. Later 45 joints were cut in crushed stone concrete on F.A. Route 135, Section 10, and F.A. Route 161, Section 10-H. Results here were decidedly better than in gravel concrete. Less time was required to make the cuts, less wheel wear and breakage occurred, and better cuts were obtained.

All of the cut joints were examined in 1941 by two members of the University committee. In general they found these joints to be in good condition, with considerably less spalling of the edges than had occurred at joints installed and finished in the orthodox manner.

It seems quite probable that spalling can be still further reduced by improvements in the jointing and cutting technique. Rounding off the sharp edges of the joint, as shown in Fig. 134a, should make these edges less susceptible to traffic abrasion and impact. This could be done by adding another trailer wheel of appropriate shape to the machine, or by other means.

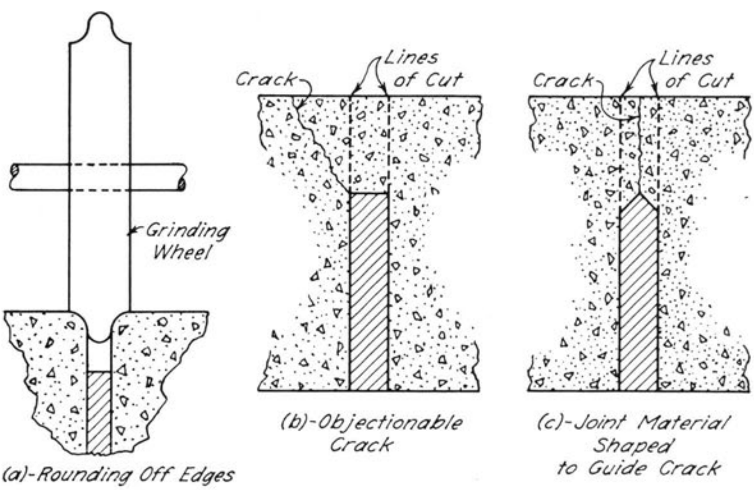


FIG. 134. POSSIBLE IMPROVEMENTS IN PROCESS FOR CUTTING JOINTS

Although it did not occur at any of the joints included in the investigation, there is a possibility that the shrinkage crack over the joint material may sometimes fall outside of the joint space, as shown in Fig. 134b, a defect which would leave spalled edges along the joint. The tendency for such objectionable cracks to form would be reduced by early cutting, and might be reduced by shaping the top of the joint material as shown in Fig. 134c.

It should be pointed out that these results were accomplished with the first machine developed for field use. This machine had a cutting range of only 11 ft. and hence had to be set up twice to cut a joint across a full-width pavement. A properly designed machine which would permit cutting a complete joint with one setting would undoubtedly produce better results and greater efficiency.

23. *Summary*.—It is true that from the standpoint of production, outstanding results were not obtained in the field during this investigation. However, in the development of any new process, efficiency is not expected in initial results. Early work, such as was conducted in this investigation, can only indicate the feasibility of a process and serve to guide others who may attempt to improve the methods and equipment.

It is believed that the principle upon which this method is based is fundamentally sound, and that the results of the investigation, particularly those obtained in the laboratory, indicate that the method has possibilities for practical development.

VII. Conclusions

24. *Conclusions.*—As a result of the investigations and studies reported in this bulletin, the following conclusions, based entirely on data obtained in Illinois, are believed warranted. Conclusions which are based on field investigations apply only for the particular climatic, soil, and other conditions under which the data were obtained and to the specific types of joints included in the investigations. The conclusions should be considered in relation to each other whenever a relationship exists. The various types of joints and load transmission devices included in these investigations were installed and tested at the request of their manufacturers.

(a) General Conclusions

The conclusions in this group pertain to all the types of joints covered by this bulletin:

(1) Joints for concrete pavements are still in the development stage, and no joint or system of joints has been developed which meets all the essential requirements of a satisfactory joint.

(2) Spalling of the edges of the concrete at joints and at the junction of transverse and longitudinal joints, particularly the latter, is an objectionable condition prevailing to a considerable degree at all types of expansion and contraction joints. Honeycombed and poorly compacted concrete under the flanges of the seals on metal joints contributes to spalling at these joints. Improper edging of joints and the poor quality of concrete at joints, resulting from conventional methods of installation, also contribute to spalling.

(3) All joints whose installation requires the finishing machine to be stopped and the screed raised at the joint are likely to have inferior concrete adjacent thereto. This condition, and the irregular surface resulting from hand finishing, tend to promote spalling of the edges along the joints.

(4) The method of installing joints below the pavement surface, pouring the slabs monolithically, and cutting down to the joint material with an abrasive wheel after the concrete has hardened offers some possibility for improving both the quality of the concrete adjacent to joints and the riding quality of pavements.

(5) None of the expansion joints included in this investigation prevents soil from the surface or the subgrade entering the joint space. Accumulations of soil were found in all types of joints.

(6) All joints included in this study permit percolation of water to the subgrade. The air-chamber joints, once the seals split, appear

to be the worst in this respect, followed by the fiber joint. The bituminous premolded and the wood joints appear to be the most effective in restricting the passage of water.

(7) All types of joints included in the Armington Experimental Road can be well installed with proper equipment and reasonable care. Metal-sealed, air-chamber joints are more difficult to install than the premolded types. Wood joints are far superior to any other type from this standpoint.

(b) Copper Sealed Joints

The most significant and positive conclusions resulting from these investigations are those concerning the behavior of joints with copper seals designed to prevent water and foreign material entering the joint from the top and ends. These conclusions are as follows:

(8) One of the most serious defects of metal air-chamber joints is the extensive failure of the copper seals, which renders the joints ineffective against the flow of water to the subgrade and against the infiltration of foreign material into the expansion space.

(9) Laboratory tests showed that some seals were more durable than others, but the difference was only relative. All of the seals developed splits early in the accelerated flexing test, indicating that none possessed the durability required by economic considerations.

(10) Laboratory tests, supplemented by field observations of joints in service, indicate that none of the copper seals investigated offers any assurance of remaining effective more than a few years. Since the copper seal is a vital and expensive feature of the air-chamber joint, the use of such joints is not warranted.

(11) Copper seals on premolded fiber joints fail after only a few years' service, and therefore their use also is not economically justified.

(12) Copper seals on metal contraction joints are more durable than those on expansion joints. However, it appears that they also do not have sufficient effective life to warrant their use.

(13) An important factor which contributes to the failure of copper seals is the vertical force exerted on the seal by traffic pounding on the cap and inert material which works into the space above the seal. Another important factor appears to be the differential vertical movement between the transverse edges of the slabs adjacent to the joint, as a load passes across the joint. Vibration of the ends of both slabs may also influence the cracking of copper seals. Fatigue appears to be an important factor in the failure of copper seals.

(14) In spite of conflicting evidence, it appears that the volume and weight of traffic affect the life of copper seals, but, even where

subjected to relatively light traffic, copper seals do not have a life expectancy which justifies their cost.

(15) From the laboratory tests, it may be concluded that during installation an undesirable amount of concrete or mortar may enter air-chamber joints, thus forming an obstruction to complete closure. The pressure exerted on the walls of these joints by the plastic concrete may produce an appreciable variation in joint width, so that the full designed provision for expansion may not be secured.

(c) Closing of Expansion Joints

The major function of expansion joints is to provide and maintain spaces into which the slabs can expand and thus prevent blowups. Many factors tend to decrease or nullify the effectiveness of joints in this respect. The following conclusions relate to this function of expansion joints:

(16) Infiltration of foreign material into cracks and joints, an important factor in the progressive closing of expansion joints, leads to blowups. It appears that air-chamber expansion joints, installed at 90-ft. intervals with two intervening contraction joints in nonreinforced pavement, will close at an average rate of about 0.1 in. per year, but individual joints may close at a much higher or lower rate.

(17) The rate of permanent closure of expansion joints is a function of panel length, number of contraction joints and transverse cracks, and type of expansion joint, the most rapid closure occurring at joints which provide little resistance to compression.

(18) Contraction joints and transverse cracks in nonreinforced pavements with expansion joints show permanent opening, the movement being absorbed by the expansion joints.

(19) Accumulations of soil in joints decrease the available expansion space by reducing the free opening of air-chamber joints and decreasing the thickness of premolded fillers. This reduces the length of time that joints will be effective in preventing blowups. The effect of concentrated loads caused by soil in the ends of joints is already apparent from the many failures of the extreme corners of the concrete slabs adjacent to joints.

(20) Expansion joints cannot fulfill their purpose of preventing blowups if foreign material enters them, or if contraction joints and transverse cracks collect foreign material during periods of contraction, which causes a progressive growth in the length of pavement between expansion joints.

(21) Wire mesh in Illinois pavements built in 1938 was still effective in holding transverse cracks tightly closed and in preventing

pavement growth from that cause after seven years of service. No prediction as to the ultimate serviceable life of mesh can be made from these investigations.

(22) All joints open and close with temperature changes, both seasonal and daily, the movements due to the latter being small. The amount of movement at expansion joints is related to spacing between expansion joints, the number of intervening contraction joints and transverse cracks, and the resistance offered by the joint and its load transmission devices and the subgrade.

(d) Premolded Caps and Top Fillers

The air-chamber expansion joints included in these investigations were provided with a premolded cap or a hot-poured bituminous filler over the metal seal to protect it and to seal the space above it against the entrance of foreign material. The top of the premolded joints and contraction joints was filled with a hot-poured bituminous filler to prevent foreign material and water entering the joint from the top. Experience from these investigations has shown that:

(23) The effective life of premolded caps such as those used on the metal joints is very short, probably not more than one or two years.

(24) The premolded caps and poured top fillers on the expansion and contraction joints included in this bulletin definitely are not effective in preventing infiltration of water and foreign material.

(25) In cool and cold weather, the top surface of the filler on all types of expansion joints except wood, when maintained as in Illinois, will in general be lower than the surface of the surrounding pavement. Except at 4-in. open joints, the amount of intrusion will not be sufficient to produce noticeable vertical displacement of a vehicle. The average depression at 4-in. open joints ranged from ¼ to ½ in., which on wide joints is sufficient to be noticeable, if not disturbing, to passengers in a vehicle.

(26) Even in cold weather, some high fillers will be found at expansion and contraction joints maintained as in Illinois. These can be expected to be comparable to the extrusion or piling of filler at transverse cracks, and should contribute no more to roughness, except as their influence may be intensified at certain speeds by rhythmic build-up of the spring movements of vehicles due to the regular spacing of joints.

(e) Premolded and Wood Joints

The following conclusions are based on the experience in Illinois with premolded joints and joints made of wood boards:

(27) The data are not sufficient to establish the serviceable life of premolded fiber and bituminous premolded joint fillers. While they possess certain defects which are bound to reduce their life, they are more economical and practical than air-chamber joints.

(28) Joints made from cypress boards appear to possess several very desirable advantages over other types of joints and show promise of being the best type of joint available at present. However, experience with wood joints in Illinois is extremely limited.

(f) Spacing of Joints

Opinions among highway engineers as to the proper spacing of joints vary widely. Some recommend that joints be installed at 10-ft. intervals; others are of the opinion that no joints should be used or that they should be installed at long intervals. The following conclusions relative to joint spacing are indicated from the experience in Illinois:

(29) If appropriate consideration is given to the age of the pavements, there are indications that Illinois pavements built with joints at approximately 1,000-ft. intervals are better riding and better appearing than those built with closely spaced joints.

(30) Since frequency and regularity of surface variations affect riding qualities adversely, joints should be installed at the longest interval commensurate with other design requirements.

(31) There were almost as many transverse cracks per mile, in addition to joints, in pavements four years old with joints at 30-ft. intervals as in pavements of the same age with 4-in. open joints at intervals of 800 to 1,000 ft.

(32) An examination at the end of three years of the subsections on the Armington Experimental Road, with joints at from 15- to 50-ft. intervals, showed a tendency for the number of natural transverse cracks per mile to decrease as the length of slab panel decreased. In every case, however, the shorter the original panel length, the greater was the combined number of joints and natural cracks per mile. Since a joint is nothing more than a controlled crack, introducing the same weaknesses into a pavement that natural cracks do, the installation of joints at short spacings to reduce or eliminate transverse cracking may be undesirable.

(g) Cracking of Slabs

One of the reasons for installing joints is to control transverse cracking. The following conclusions relative to the formation of transverse cracks and corner breaks are indicated from the results of the investigations reported in this bulletin:

(33) Installation of joints at 30-ft. intervals will not prevent or retard the occurrence of transverse cracks. There is no indication that the type of joint influences cracking of slab panels.

(34) Cracking increases with age, but the rate of increase from year to year is not uniform.

(35) A relatively smaller number of transverse cracks was found in pavements which follow the existing grade closely than in those on fills, in cuts, or at the transition from cut to fill. There was little difference in the cracking of pavements in the last three classifications.

(36) The high frequency of broken panels over culverts suggests that steps should be taken to determine the cause or causes of this abnormal cracking and the remedy thereof.

(37) Corner failures have not occurred with sufficient frequency in any of the pavements covered by these investigations to be of serious consequence. Failure at interior corners occurred more frequently than at exterior corners.

(h) Load Transmission Devices

Only a cursory examination of load transmission devices was made during the field investigations, it being impractical to make extensive examinations. The following conclusions relative to these joint accessories are based largely on the results of laboratory tests:

(38) The limited data relating to load transmission devices obtained from the field investigations do not justify any definite conclusions as to the relative performance of various types of devices.

(39) Laboratory tests emphasize the importance of increasing the bearing area between the load transmission device and the concrete.

(40) Load transmission devices should be made of such material and design that they will not become ineffective due to corrosion during the service life of pavements.

(41) Load transmission systems which employ a continuous plate dowel promote concrete failures along the joint, due to the prying action of the plate dowel on the ledges of concrete above and below the horizontal slot formed by the plate. This type of failure may be accentuated by improper installation.

(42) Laboratory tests indicate that the aggregate interlock provided by the irregular faces of a transverse crack is an effective means of transferring load, as long as the crack opens only a small amount.

Hence, a type of construction which will hold the two sides of a transverse crack in close contact appears desirable. Tests indicate that wire mesh reinforcement is effective in preserving the initial load transmission properties afforded by aggregate interlock, as long as the mesh remains intact.

(43) Load transmission devices add to construction problems, since extreme care and close supervision are required to obtain proper alignment of these accessories. It is particularly difficult to maintain load transmission devices in good alignment with joints of soft material such as rubber and bituminous premolded materials. Wood joints are very effective in this respect.

(i) Dummy Transverse Joints

During the period covered by these investigations, experience of the Division of Highways with dummy transverse joints was limited to those installed in the Armington Experimental Road. The following conclusions are based on experience with joints 1 to 2½ in. deep:

(44) It appears that a dummy transverse joint 1 in. deep in a 9-in. - 6½-in. - 9-in. pavement was sufficient to produce proper cracking of the slab, but a 1½-in. depth was the easiest to construct.

(45) No apparent faulting has occurred at the dummy joints on the Armington Experimental Road.

(j) Ability of Pavements to Resist Compression Influenced by Joints

Laboratory tests yield evidence that joints produce concentrations of stress in the face of the concrete along the joint, when the joint becomes tightly closed. This conclusion is inferred from the fact that test specimens containing a section of a joint developed concrete failures at loads considerably less than the theoretical load based on the cross sectional area of the specimen and the unit compressive strength of the concrete from which the specimens were made, as determined from tests of 6-in. x 12-in. cylinders. These concentrations appear to be caused by the separators in the air-chamber expansion joints which stiffen the side walls and hold them apart, and by the variation in the compressibility of the premolded joint materials. The following conclusions are indicated by the laboratory tests:

(46) Test specimens containing air-chamber and fiber joints both failed under unit compressive loads which were lower than the unit compressive strength of the concrete in the specimens. It seems probable that a section of road slab containing a tightly closed joint would

be less resistant to longitudinal crushing forces due to expansion of the pavement than the unbroken plain concrete slab.

(47) Premolded and poured joints were more effective than the air-chamber joints in distributing compressive stresses after the joints had become tightly closed.

(k) Pavement Surface Smoothness Affected by Joints

A common criticism of joints is that they affect the riding quality of a pavement because of surface irregularities introduced by high or low fillers and by variations in the concrete surface itself due to curling or poor construction. The size of the irregularities in the concrete surface was determined by measuring ordinates to the pavement surface from a 10-ft. straightedge placed along the normal wheel paths of the pavement. The portion of the pavement surface considered in this investigation was the 10-ft. length bisected by the joint. The following conclusions are based on the data thus obtained:

(48) For each type of joint the average maximum variation of the concrete surface included within the 10-ft. length bisected by the joint was for the most part only slightly greater than the limiting value for surface variation permitted by the specifications in effect at the time of construction.

(49) On the basis of a roughness rating which gives consideration to both the number and relative size of variations exceeding ⅛ in. in 10 ft., the pavement surface included within the 10-ft. length bisected by a joint was in general somewhat smoother at air-chamber expansion joints than at other types of expansion joints. Roughness ratings indicated smoother surfaces at metal contraction joints than at air-chamber expansion joints.

(50) Pavements with joints of any type have variations in the concrete surface greater than those permitted by the specifications and above the limits universally accepted for good riding qualities. This indicates that surface roughness is induced by the presence of joints.

(51) Concrete surfaces adjacent to joints in pavements of the ages covered by this bulletin are subject to greater variations in roughness than those at transverse cracks, and jointed pavements can be expected to be somewhat less smooth riding than unjointed pavements.

(52) Because the number of joints and transverse cracks in a pavement with joints at close intervals exceeds for many years that for a pavement without joints or with joints at long intervals, the riding quality of the former may be expected to be poorer than that of the latter for an important part of the service life of the pavement.

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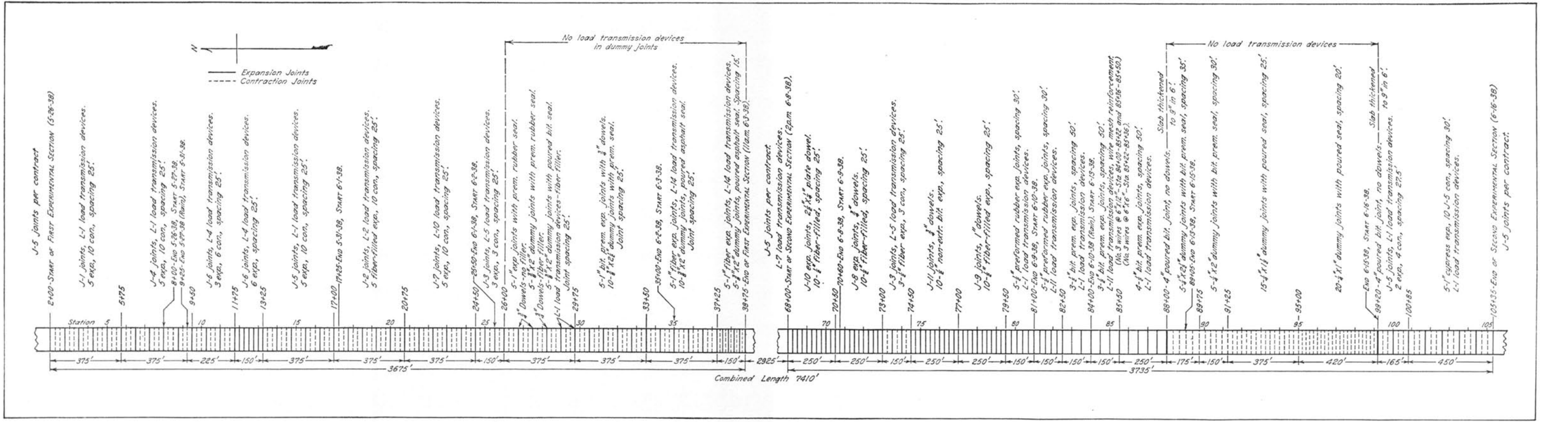

Fig. 92. Layout of Armington Experimental Road

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TABLE 2

SUMMARY OF STATE HIGHWAY DEPARTMENT SPECIFICATIONS AND PRACTICE IN DESIGN OF PORTLAND CEMENT CONCRETE ROADS

(Published through the courtesy of the American Iron and Steel Institute)

State	General Type of Cross Section: Thickness, in.	Method of edge thickening	Lane widths, ft.: Single	2-lane	3-lane	Reinforcement, Distributed: Type specified	Weight per 100 sq. ft., lb.: Loose bars or bar mats	Wire mesh or expanded metal	Where required	Design: Marginal bars No.	Size	Hairpin corner bars No.	Size	Other types No.	Size	Longitudinal Joints, Types: Dummy (Grooved or ribbon)	Metal plate	Construction (Plain or deformed)	Tie Bars: Size	Spacing	Length
Ala.	7 uniform	None	None	10-11	10-11	Bar mats or mesh	48.9	44.7	All conditions	...	....	...	....	...	....	Grooved or ¾″ x 2″ premolded	......	Deformed	½″ φ	2′-6″ c.c.	2′-6″
Ariz.	9-7-9	2′ Taper	None	11	11	None	None	None	At acute angles only	...	....	...	....	...	....	......	Deformed	Plain	½″ sq.	2′-6″ c.c.	2′-6″
Ark.	9-7-9	Parabolic	9	9-11	11	Mesh	Bars only on bridge approaches	49.0	All conditions	2	⅝″ rnd. each edge	...	....	...	....	......	Hinged joint 16 gage	....	½″ φ	5′-0″ c.c.	....
Cal.	11-8-11 9-7-9 9-6-9	2′ Taper	None	10-11	11-12	None	None	None	Bars on bridge and culvert app.	...	....	...	....	...	....	None	None	Plain	For special cases ¾″ φ	4′-0″ c.c.	2′-0″
Col.	Practically all roads are oil-surfaced					.....	.....	....	....	...	....	...	....	...	....	......	......	....	....	.....	....
Conn.	8 uniform	None	None	11-12	11-12	Bar mats or mesh	62.13 ave.	61.27 ave.	All conditions	...	....	...	....	...	....	None	None	Deformed	½″ φ	2′-6″ c.c.	2′-6″
Del.	8 uniform	None	10-11	10-11	None	Mesh	None	54.0	Heavy traffic conditions	...	....	...	....	...	....	None	Deformed	Plain	½″ φ	5′ c.c.	4′-0″
D.C.	8 uniform 6 uniform	Integral curb	None	As shown on plans		Mesh	None	50.0 30.0	All conditions	...	....	...	....	...	....	Subject to approval	Deformed	Plain	½″ φ	2′-6″ c.c.	3′-0″
Fla.	7 uniform 9-7-9	3′ Taper	None	11	10	Bar mats	60.0	None	Unstable sub-grade or heavy traffic	...	....	...	....	...	....	Ribbon	Deformed	Deformed	½″ φ	5′ c.c.	4′-0″
Ga.	8½-6½-8½	3′ Taper	None	9-12	11-12	None	None	None	Bars on bridge approach only	...	....	...	....	...	....	Grooved or ribbon	Deformed	Plain or deformed	½″ φ	2′-6″ c.c.	3′-0″
Idaho	10-7-10	2′ Taper	None	12	12	None	None	None	Over inadequate subgrades	...	....	...	....	...	....	Grooved	None	Plain	¾″ φ	5′ c.c.	4′-0″
Ill.	10-8-10 9-7-9	2′ Taper	10-12	10-12	11-12	Mesh or expanded metal	None	54.0	All conditions	...	....	...	....	...	....	None	Deformed	Deformed	½″ φ	2′-6″ c.c.	2′-6″
Ind.	9-7-9 9-6-9	2′ Taper	15	11	11	Mesh	None	45.0 36.0	All conditions Bars (225 lb.) at br. apprs.	...	....	...	....	...	....	Grooved or ribbon	Deformed	Deformed	⅝″ φ	5′ c.c.	4′-0″
Iowa	10-7½-10	3′ Taper	None	10	10-11	None	None	None	See note under "Design Reinf."	6	¾″ rnd. See "A"	...	....	...	....	Ribbon	None	Deformed	½″ φ	2′ c.c.	4′-0″
Kan.	10-8-10 9-7-9 8-6-8	Parabolic	None	11	11	Mesh	None	42.0	All conditions	...	....	...	....	...	....	Grooved	None	Deformed	½″ φ	2′-6″ c.c.	3′-0″
Ky.	9-7-9 9-6½-9	2′ Taper	None	11	11	Mesh or expanded metal	None	43.0	All conditions	...	....	...	....	...	....	Grooved or ribbon	Deformed	Deformed	½″ φ	5′ c.c.	4′-0″
La.	9-6-9	3′ Taper	None	12	12	As determined by P.R.A.										Grooved	None	Deformed	½″ φ	2′-6″ c.c.	2′-0″
Maine	9-7-9	Parabolic	None	11-12	11-12	Bar mats	120.0	None	All conditions	...	....	...	....	...	....	None	None	Plain	⅝″ φ	3′-4″ c.c.	4′-0″
Md.	10-7-10 9-7-9 9 uniform	Parabolic	10	12	12	Bar mats or mesh	64.0	60.0	Heavy duty roads	...	....	...	....	...	....	None	None	Plain or deformed	⅝″ φ	4′ c.c.	....
Mass.	8 uniform	None	None	11-12	11-12	Bar mats	52.0	None	All conditions	...	....	½″ sq. x 6′-0″		...	....	None	None	Plain	½″ sq.	5′ c.c.	4′-0″
Mich.	9-7-9 10-8½-10	3′ Taper	None	11	11	Bar mats or mesh	60.0	60.0	All conditions	...	....	...	....	...	....	Ribbon	None	Plain	½″ φ	3′-4″ c.c.	4′-0″
Minn.	9-7-9 9-6-9	4′ Taper	None	10-12	11-12	None	None	None	Bars for special conditions only	Some marginal bars used		...	....	...	....	None	None	Plain	½″ φ	2′-6″ c.c.	2′-0″
Miss.	9-6-9 7-5-7	3′ Taper or Parabolic	None	10-11	None	Bar mats or mesh	64.0	55.0	Practically all conditions	...	....	...	....	...	....	Grooved or ribbon	Deformed	Deformed	½″ φ	2′-6″ c.c.	2′-6″
Mo.	9-7-9 9-6-9	2′ Taper	None	11	11-12	Bar mats or mesh	53.0	44.0	All conditions	...	....	...	....	...	....	None	Deformed	Deformed	½″ φ	5′ c.c.	3′-0″
Mont.	No concrete roads constructed since 1929					.....	.....	....	....	...	....	...	....	...	....	......	......	....	....	.....	....
Neb.	9-7-9	3′ Taper	None	11	10-11	Mesh or bar mats	42.0	42.0	With sand-gravel aggregate	...	....	8½″ rd. x 12′ at each exp. joint		⅝″ rd. x 13′ @ 3′ o.c. each side of center line		Ribbon	Deformed	Plain	½″ φ	Note "F" 5′ c.c.	4′-0″
Nev.	9-7-9	2′ Taper	10	10	10	Mesh	None	42.0	All conditions	Note: No concrete roads since 1933						None	None	Plain	....	......	....
N. H.	9-6-9 7 uniform	3′ Taper	None	12	11-12	Bar mats or mesh	75.0	66.0	All conditions	...	....	...	....	...	....	None	None	Plain	⅝″ φ	3′ c.c.	4′-0″
N. J.	9 uniform 10 uniform	None	None	10-12	10-12	Bar mats or mesh	77.0	74.0	All conditions	1	¾″ rnd. x10′ each corner	...	....	...	....	None	None	Plain with ¾″ joint filler and metal flashing	....	......	....
N. M.	9-6-9	2′ Taper	None	10	None	Bar mats or mesh	34.0	34.0	All conditions	...	....	...	....	...	....	Grooved with reinf. slabs	Deformed with un-reinf. slabs	Plain	¾″ φ	5′ c.c.	2′-0″
N. Y.	8-7-8 8 uniform	Tapered	None	10-12	10-12	Bar mats or mesh	60.0 See Note "B"	60.0	All conditions	...	....	...	....	...	....	Tongue and groove	None	Plain	¾″ bolt	5′ c.c.	7″-12″
N. C.	8 uniform 9-7-9	2′ Taper	None	11	11	None	None	None	....	...	....	...	....	...	....	Grooved or ribbon	Deformed	None	½″ φ	2′-6″ c.c.	2′-0″
N. D.	9-7-9	2′ Taper	11	11	None	Mesh	None	57.77	All conditions	...	....	...	....	...	....	Grooved	None	None	½″ φ	2′-6″ c.c.	2′-6″
Ohio	8-10 uniform 7-9 uniform	None	None	10-12	12	Bar mats or mesh	54-58	54-58	All conditions	...	....	...	....	...	....	Grooved or ribbon	Deformed	Plain	⅝″ φ	4′ c.c.	4′-0″
Okla.	9-7-9 9-6-9	3′ Taper	None	11	12	Mesh	None	60.0	For city streets and unstable fills	...	....	...	....	...	....	None	None	Tongue and groove	½″ φ	2′-6″ c.c.	2′-6″
Ore.	8 uniform	None	16	12	10-12	None	None	None	....	...	....	...	....	...	....	None	None	Deformed	½″ sq.	3′-6″ c.c.	3′-0″
Pa.	9-10—8-9 uniform	None	None	11-12	11-12	Bar mats or mesh	45-48	45-48	65 lb. in special cases	...	....	...	....	...	....	Poured cleft or pre-molded cleft	None	Plain	½″ φ	5′ c.c.	....
R. I.	8 uniform 7 uniform	None	None	11-12	11-12	Mesh	None	46.0	All conditions	...	....	Yes		...	....	None	None	Plain	....	.....	....
S. C.	8-6-8	3′ Taper	None	11	10	Bars or mesh	½″ rnd. @ 12″ o.c. each	56.0	At bridge appr. or unstable subgrade	...	....	...	....	...	....	Grooved or ribbon	Deformed	Plain or deformed	½″ φ	2′-5″ c.c.	2′-0″
S. D.	9-6-9	4′ Taper	None	10	None	Mesh	None	42.0	All conditions	...	....	...	....	...	....	Grooved	Deformed	Plain	½″ φ	2′ c.c.	4′-0″
Tenn.	8 uniform 8-6-8	Parabolic	None	11	11	None	None	None	On new sub-grades only	...	....	...	....	...	....	Grooved or ribbon	Deformed	Plain	½″ φ	2′-6″ c.c.	....
Texas	9-7-9 8-7-8	3′ Taper 4′ Taper	None	10-12	10-12	Bar mats	25.5-30.5	None	For bridge approaches only	See Note "C"		4	½″ rd. x 6′-0″ long	...	....	Ribbon	Deformed 18 gage metal	Deformed	½″ φ	2′-4″ c.c.	....
Utah	9-7-9	2′ Taper	None	11	None	.....	.....	....	....	...	....	...	....	...	....	......	......	....	....	.....	....
Vermont	7 uniform	None	None	10	None	Bar mats	84.89	None	All conditions	...	....	...	....	...	....	None	None	Plain	⅝″ φ	2′-10″ c.c.	4′-0″
Va.	9-7-9	3′ Taper	None	11-12	11	Mesh or expanded metal	None	44-58	All conditions	...	....	...	....	...	....	None	Deformed	Deformed	½″ φ	2′-6″ c.c.	2′-6″
Wash.	10-7-10	2′ Taper	11	11	11	None	None	None	Bar mats at br. appr. and over culverts	...	....	...	....	...	....	Grooved	None	Deformed	½″ φ	2′-2″ c.c.	2′-0″
W. Va.	10-6.7-10 9-6-9	2′ Taper	None	10-11	11	Mesh	None	43.0	All conditions	...	....	...	....	...	....	None	None	Deformed	½″ φ	5′ c.c.	3′-6″
Wis.	10-8-10 9-6½-9	2′ Taper	None	10-11	10-11	Mesh	None	58.0	Dependent on subgrade	...	....	...	....	...	....	Grooved or ribbon	Deformed	Deformed or plain	½″ rnd. ½″ sq.	3′ c.c. 4′ c.c.	3′-4″
Wyo.	6 uniform 9-6-9	2′ Taper	None	15	None	Bar mats	89.07	None	All conditions	...	....	...	....	...	....	None	None	None	....	.....	....

State	Transverse Joints, Expansion: Spacing ft.	Width in.	Types of joint filler required or allowed	Contraction, Dummy: Type	Depth in.	Metal plate	Spacing ft.	Method of Load Transfer, Dowel bars: Size	Spacing	Other approved types: At expansion joints	At contraction joints	Unreinforced Pavements: Were unreinforced pavements constructed since W.P.B. restrictions?[1]	Total length to date, miles	Intention to continue design and construction of unreinforced pavements	Expect to return to use of distributed reinforcement after emergency	Wt./100 s.f. to be specified: Bar mats or bars	Mesh or expanded metal
Ala.	40	1	Premolded or poured bituminous or cotton seed hulls	Grooved	2	None	40	¾″ φ x 1′-3″	15″ c.c.	Several patented devices as alternates		Yes	17.12	Doubtful	Yes	48.9	44.7
Ariz.	600	¾	Redwood	....	¼t	Plain	12′-6″	¾″ φ x 1′-0″	15″ c.c.	10′ Tapered ends	None	Before and during	10.0	Yes	No	......	......
Ark.	50	¾	Poured mastic	....	....	None	....	¾″ φ x 2′	....	Type with ¾″ x 4½″ c.r. pin	None	Yes	44.0	No	Yes	At bridge ends	49 to 90
Cal.	120 to 240	¾	¾″ Redwood	Grooved	2	None	15 to 20	¾″ φ x 1′-2″	15″ c.c.	None	None	Before and during	......	Yes	No	......	......
Col.	......	....	....	....	....	....	....	......	....	......	......	......	......	.....	.....	......	......
Conn.	97′-4″	½	Premolded bit. to 1¼″ of surface; poured bit. to ¾″ of surface	Grooved	1½	None	24′ to 25′-4″	None	....	Special beam 1½″ x 12″ @ 19″-21″	None	Yes	79,410 cu. yd.	No	Yes	62.12 ave.	61.27 ave.
Del.	90	¾	Rigid joint with cork or other filler	None	....	Plain	30	¾″ φ x 2′	12″ c.c.	Rigid assembly joints		Before and during	Most of mileage	Will use both types		......	54.0
D.C.	30	½	70% recovery material sealed 2″—90% sealed ½″	None	....	Plain	12½ for bases	¾″ φ x 1′-3″	12″ c.c.	As approved		Yes	13.0	No	Yes	......	50.0 to 30.0
Fla.	60	¾	Cork and rubber. Rubber or cotton seed hulls	Grooved	2	None	20	¾″ φ x 2′	12″ c.c.	Translode	Crosslode	Yes	26.2	Yes	Where conditions require	60.0	......
Ga.	90 to 120	¾	Any approved non-extruding type	Grooved	¼t	None	20-30	¾″ φ x 2′	12″-15″	10 alternate types		Before and during	Most of mileage	Yes	No	......	......
Idaho	90	¾	Premolded	Grooved	2	Plain	15	¾″ φ x 2′	12″	None	None	None	Most of mileage	Yes	Where condition requires	......	......
Ill.	50	¾	Premolded fibre, cork rubber or cork-rubber	None	....	....	....	¾″ φ x 2′	13½″	Wing anchor bars and Translode	None	Yes	......	No	Yes	......	Not determined
Ind.	120	¾	Fibre, rubber, cork or cork-rubber	Grooved	¼t	Plain or deformed	40	¾″ φ x 2′	12″	Other approved load transfer units		Yes	91.55	No	Yes	At bridge approaches	Not determined
Iowa	120	1	Bituminous	¹⁄₁₆″ x 3″ bit. ribbon	3	None	30	¾″ φ x 2′	12″	None	None	Yes	12.41	No	Yes	......	......
Kan.	120	1	Premolded	Grooved	2	None	20	None	....	Concrete pavement support, 9″ x 1′-6″		......	......	No	Yes	......	42.0
Ky.	120	1	Premolded non-extruding or metal	Grooved	2½	Plain	30	¾″ φ	12″	Approved load transfer units		Yes	30.0	Not decided	Not decided	Not decided	40-45
La.	120	¾	Fibre, cypress or redwood	Grooved	1½	Plain	20	¾″ φ x 1′-6″	15″	None	None	Yes	26.77	To be determined by P.R.A.			
Maine	40	½	Premolded cork, cor-rubber, bituminous, fibre	None	....	None	....	¾″ φ	12″	None	None	None	......	No	Yes	120.0	......
Md.	120	1	Cork or rubber (non-extruding)	Grooved	2	None	15	¾″ φ	12″-15″	Approved patented devices		Yes	20.0	For secondary roads	Yes	64.0	60.0
Mass.	51′-2″	½	Cork, sponge, rubber premolded bituminous	None	....	None	....	¾″ φ	15″	As approved		Yes	2.1	No	Yes	52.0	......
Mich.	120	1	Non-extruding	Grooved	2½	None	30	¾″ φ	15″	J-Bar and Tee-Gee	Dowel bars	Yes	200.01	No	Yes	60.0	60.0
Minn.	120	1	Non-extruding	Grooved	3	None	15-20	⅞″ φ	15″	None	None	Before and during	15.0	Yes	No	......	......
Miss.	40	¾	Premolded with ½″ poured seal; poured bitum.; bit. cotton seed hulls	None	....	None	....	¾″ φ	15″	None	None	Yes	9.11	Not decided as yet			
Mo.	40 (Reinf.) 20′-25′ (Unreinf.) (note "D")		Premolded bituminous or wood fibre	Grooved	t-½″	None	See note "E"	⅞″ φ x 16″	12″	Translode or transmission angles		Yes	25.11	Not fully determined			
Mont.	......	....	....	....	.....	....	....	......	....	......	......	......	......	.....	.....	......	......
Neb.	120	1	Premolded or poured	Grooved	2½	None	30	¾″ φ x 27″	16½″	Devices as approved		Yes	14.2	No	Yes	42.0	42.0
Nev.	50	½	Cork, rubber or fibre	Grooved	2	None	50	¾″ φ x 2′	36″	None	None	None	......	No	Yes	......	42.0
N. H.	50	¾	Non-extruding types	None	....	None	....	¾″ φ	12″	Beth. or equal	None	None	......	No	Yes	75.0	66.0
N. J.	56′-4″	¾	Premolded bituminous	None	....	None	....	2″ x ½″ x ¾″ channels	12″	None	None	Yes	14.4	No	Yes	77.0	74.0
N. M.	60	¾	Premolded bituminous	Grooved	2½	None	30	¾″ φ x 2′	12″	None	None	None	......	No	Yes	34.0	34.0
N. Y.	89′4″-98′	¾	Premolded bituminous or wood	None	....	None	....	None	....	Malleable castings or H-beams	None	Yes	Slight mileage	No	Yes	60.0	60.0
N. C.	120	¾	Rubber, cork or pre-molded fibre	Grooved	2	None	30	¾″ φ	15″	Dow-weld or Translode	Metal	Before and during	25.37	Yes	No	......	......
N. D.	100	1	Cork, sponge, rubber, cork-rubber or self-expanding cork	Grooved	1½	Deformed	25	¾″ φ x 2′	12″	None	None	None	......	No	Yes	......	57.77
Ohio	120	¾″-1″	Premolded bitum. or with steel extrusion pls. or self-expanding cork	Grooved	½t-¾″	None	60	¾″ φ x 15″	15″	Translode base permitted		Yes	97.8	No	Yes	54-58	54-58
Okla.	90	¾	Premolded bituminous	Grooved	2½	None	30	¾″ φ x 2′	12″	U-bar; J-bar; Star-lug; or Crosslode	Bars spaced with ¾″ φ x 15″ @ 15″ c.c.	Yes	28.28	No	Yes	......	60.0
Ore.	120	¾	Treated fibre	Grooved	2	None	15	¾″ φ x 2′	12″	None	None	Before and during	......	Yes	No	......	......
Pa.	77	See note "G"	Cork, cork and rubber comp. or fibre	None	....	None	....	1″ φ	12″	Dow-weld, Beth., and Five-way	None	Yes	113.1	No	Yes	45-48	45-48
R. I.	73′-6″	¾	Cork or expanding cork	Grooved	2½	None	20	¾″ φ x 2′	12″	Approved types	None	Yes	3.7	No	Yes	......	46.0
S. C.	90	1	Cotton seed hulls and cutback asphalt	Ribbon	¼t	Plain	30	¾″ φ x 1′	15″	Flange angle type	Metal optional	Yes	17.4	Yes	No	......	56.0
S. D.	90	1	Premolded	Grooved	....	None	30	¾″ φ x 2′	12″	None	None	None	......	No	Yes	......	42.0
Tenn.	90-120	¾	Premolded fibre	Grooved	2¾	Plain	30	¾″ φ	12″-15″	None	None	Before and during	......	Yes	No	......	......
Texas	120	½″-1″	As called for on plans	Grooved	2	None	15	¾″ φ x 1′-9″	14″	None	¾″ φ dowels x 1′-0″, 4½″ below surface	Yes	Not determined				
Utah	......	....	....	....	....	....	....	......	....	......	......	......	......	.....	.....	......	......
Vermont	55′-4″	½	Premolded cork or other non-extruding type	None	....	None	....	¾″ φ x 2′	12″	None	None	None	......	No	Yes	84.89	......
Va.	90	¾	Premolded bituminous with ½″ bitum. seal or sponge rubber	Grooved	2½	Plain	30	¾″ φ x 1′	12″	Various patented devices		Yes	71.8	No	Yes	......	44-58
Wash.	120	¾	Non-extruding	Grooved	2	None	15	¾″ φ x 1′-1½″	12″	None	None	Before and during	......	Yes	No	......	......
W. Va.	120	¾	Premolded bituminous cork or rubber	Plain	2	None	20	¾″ φ x 2′	12″	None	None	Yes	24.84	No	Yes	......	43.0
Wis.	120	¾	Bituminous, fibre or wood	Grooved	t/2-¾	None	20	¾″ φ x 2′	15″	Wing anchor permitted		Yes	40.0	Yes	In some cases	......	58.0
Wyo.	30-50	¾	Cork or sponge rubber	None	....	None	....	¾″ φ x 2′	12″-18″	None	None	None	......	None	Yes	89.07	......

Note "A" — One bar 9 in. from each edge; two bars each side of longitudinal center joint, 9 and 21 in. from center line.

Note "B" — Weight per 100 sq. ft. at each slab joint is 120 lb.

Note "C" — Four ½-in. round longitudinal bars located 9 in. and 12 in. from outside edge of slab, at center line of slab and 9 in. from longitudinal joint; also three ½-in. round transverse bars between each contraction joint.

Note "D" — Transverse joints not required with limestone aggregate. Above joint spacings apply with chert aggregate. Width of joints ⁵⁄₁₆ in.; ⅝ in. at structures and other intersections.

Note "E" — Contraction joints not permitted with chert aggregate. Spacing of contraction joints with limestone aggregate: 50 ft. for reinforced pavement; 30 ft. for non-reinforced pavement.

Note "F" — When used in connection with wire mesh or bar mats. If used without wire mesh or bar mats, ⅝ in. x 13 ft. @ 3 ft. c.c. (staggered).

Note "G" — ¾ in. with stone or gravel aggregate or 1 in. with slag aggregate.

[1] This refers to the restrictions imposed by the W.P.B. in regard to the use of steel reinforcement.

NOTE: This summary has been prepared on the basis of replies to questionnaires returned by State Highway Engineers of the various states and the District of Columbia as of April 1, 1944. Every effort has been made to insure the accuracy of the summary, but the American Iron and Steel Institute assumes no responsibility for any errors or omissions. Insofar as possible, this summary represents the standard specifications in effect in each state immediately prior to the development of the present national war emergency.

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J-5 joints per contract.